U.S. DEPARTMENT OF COMMERCE
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PB80-119399

Combined Sewer Overflow Treatment by Screening and Terminal Ponding

Fort Wayne, IN

Prepared for

Municipal Environmental Research Lab, Cincinnati, OH

Aug 79

United States
Environmental Protection
Agency

Municipal Environmental Research
Laboratory
Cincinnati OH 45268

EPA-600/2-79-085
August 1979

PB 80-119399

Research and Development

Combined Sewer Overflow Treatment by Screening and Terminal Ponding

Fort Wayne, Indiana

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TECHNICAL REPORT DATA
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1. REPORT NO. EPA-600/2-79-085	2.	3. RECIPIENT'S ACCESSION NO. PB 80 119399
4. TITLE AND SUBTITLE COMBINED SEWER OVERFLOW TREATMENT BY SCREENING AND TERMINAL PONDING - Fort Wayne, Indiana		5. REPORT DATE August 1979 (Issuing Date)
		6. PERFORMING ORGANIZATION CODE
7. AUTHOR(S) Delmar H. Prah and Paul L. Brunner		8. PERFORMING ORGANIZATION REPORT NO.
9. PERFORMING ORGANIZATION NAME AND ADDRESS City of Fort Wayne One Main Street Fort Wayne, Indiana 46802		10. PROGRAM ELEMENT NO. 1BC822 SOS #1 Task 17
		11. ~~CONTRACT~~/GRANT NO. 11020 GYU
12. SPONSORING AGENCY NAME AND ADDRESS Municipal Environmental Research Laboratory--Cin.,OH Office of Research and Development U.S. Environmental Protection Agency Cincinnati, Ohio 45268		13. TYPE OF REPORT AND PERIOD COVERED Final 4/71 - 2/76
		14. SPONSORING AGENCY CODE EPA/600/14

15. SUPPLEMENTARY NOTES
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16. ABSTRACT

A 75 MGD combined sewer overflow treatment facility was constructed to obtain plant-scale data on the effectiveness and costs of screening combined sewer overflows (CSO) by three types of fine screens. The methods of screening evaluated included fixed side hill screens, rotary centrifugal fine screening, and horizontal rotary drum fine screening. Also evaluated were the benefits to the receiving waters by pollutant removal by screening, chlorination, and ponding previously by-passed CSO. Each of 38 separate CSO events were analyzed over a 13-month period commencing in January, 1975 for 14 pollutant parameters and flow rates.

None of the screening methods studied have removal efficiencies for suspended solids, BOD$_5$, COD and nutrients that were significantly different from zero at the 95% confidence level. No effect between hydraulic loading and removal efficiency was found. The least overall cost method of screening was the fixed vertical-type screen. All the screens studied have hydraulic head requirements on the order of 2 to 10 feet static loss. The two-day terminal pond following screening was shown to be capable of meeting 30 mg/l of BOD$_5$ and suspended solids 95 percent of the days studied over the 4-year evaluation period. No perceptible effect of the addition of storm-caused CSO was found on any parameter studied. Average monthly removal of BOD$_5$ and suspended solids was 75 percent. With loading rates greater than 200 pounds BOD$_5$ and suspended solids per acre per day stable, relatively high efficiency performance was observed for the pond at hydraulic detention of 1-2 days.

Construction of this project resulted in 54 percent reduction in quantity of CSO to the river. The stream showed significant improvements in instream fecal coliform.

17. KEY WORDS AND DOCUMENT ANALYSIS

a. DESCRIPTORS	b. IDENTIFIERS/OPEN ENDED TERMS	c. COSATI Field/Group
Combined sewer, Overflows, Screening, Ponds, Chlorination	Maumee River, Fort Wayne, Indiana Lake Erie Combined sewer overflow treatment	13B

18. DISTRIBUTION STATEMENT RELEASE TO PUBLIC	19. SECURITY CLASS *(This Report)* UNCLASSIFIED	21. NO. OF PAGES 141
	20. SECURITY CLASS *(This page)* UNCLASSIFIED	22. PRICE A07-A01

☆ U.S. GOVERNMENT PRINTING OFFICE: 1979 -657-146/5466

EPA-600/2-79-085
August 1979

COMBINED SEWER OVERFLOW TREATMENT BY SCREENING AND TERMINAL PONDING

Fort Wayne, Indiana

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FOREWORD

The Environmental Protection Agency was created because of increasing public and government concern about the dangers of pollution to the health and welfare of the American people. Noxious air, foul water, and spoiled land are tragic testimony to the deterioration of our natural environment. The complexity of that environment and the interplay between its components require a concentrated and integrated attack on the problem.

Research and development is that necessary first step in problem solution and it involves defining the problem, measuring its impact, and searching for solutions. The Municipal Environmental Research Laboratory develops new and improved technology and systems for the prevention, treatment, and management of wastewater and solid and hazardous waste pollutant discharges from municipal and community sources, for the preservation and treatment for public drinking water supplies and to minimize the adverse economic, social, health, and aesthetic effects of pollution. This publication is one of the products of that research, a most vital communications link between the researcher and the user community.

The study contained herein documents the demonstration and evaluation of a 75 MGD combined sewer overflow treatment facility to obtain plant-scale data on the effectiveness and costs of screening combined sewer overflows by three types of fine screens. The overall results of the project indicate the importance of physical processes for stormwater treatment due to their adaptability to automated operation, rapid startup and shutdown characteristics, high rate operation and very good resistance to shock loads.

Francis T. Mayo
Director
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ABSTRACT

The Fort Wayne Stormwater Treatment Evaluation Project was constructed to provide plant-scale data on the effectiveness and costs of screening combined sewer overflows and to quantify the benefits to the receiving water by pollutant removal by screening, chlorination and ponding previously by-passed combined sewer overflows. Three types of screens were evaluated including fixed side hill vertical screens, rotary centrifugal fine screening, and horizontal rotary drum fine screening. Design flowrate for the Facility constructed in 1973-74 is 75 MGD. Discrete grab samples were taken and analyzed for pH, temperature, setteable solids, fecal coliform bacteria, turbidity, chemical oxygen demand, five-day biochemical oxygen demand, suspended solids, volatile suspended solids, ammonia nitrogen, organic nitrogen total phosphorus, dissolved oxygen and chlorine residual. Thirty-eight separate stormwater treatment events were analyzed over a thirteen-month period beginning in January, 1975.

Of the screening methods studied, removal efficiences for suspended solids, BOD_5, COD, and nutrients were not significantly different from zero at the 95% confidence level. No effect between hydraulic loading and removal efficiency was found. The least overall cost method of screening was the fixed vertical-type screen ($25.25/MG based on annual operations and maintenance costs). All the screens studied have hydraulic head requirements on the order of 2 to 10 feet static loss.

A dual-use, 32-acre terminal pond was shown to be capable of meeting secondary effluent standards (30 mg/l as BOD_5 and suspended solids) 95 percent of the days studied in the four-year evaluation period. No perceptible effect of the addition of storm-caused combined sewer overflows was found on any parameter studied. Average monthly removal efficiency for BOD_5 and suspended solids was 75 percent for both. Loading rates greater than 200 pounds per acre per day for both BOD_5 and suspended solids were shown to result in stable, relatively high efficiency performance for the pond at a hydraulic detention period ranging from 1.1 to 2.0 days.

The construction of the Stormwater Treatment Facility resulted in a reduction of 82 percent of the by-passing of raw, untreated combined sewer overflows to the receiving stream from the tributary area. A reduction of the quantity of raw wastewater BOD_5 reaching the river due to combined sewer overflows throughout the City of Fort Wayne was reduced by 54 percent. The stream showed improvements in instream dissolved oxygen, BOD_5, and fecal coliform bacteria. However, only the fecal coliform bacteria improvement was solely attributed to the treatment of combined sewer overflows since upstream dissolved oxygen and BOD_5 also showed similar improvements.

This report was submitted in fulfillment of Grant 11020 GYU by the City of Fort Wayne, Indiana under the partial sponsorship of the U. S. Environmental Protection Agency. This report covers a period from April 1971 to February 1976, and work was completed as of October 1977.

TABLE OF CONTENTS

	PAGE
FOREWORD	iii
ABSTRACT	iv
LIST OF FIGURES	vii
LIST OF TABLES	xi
ACKNOWLEDGMENTS	xiii
SECTION 1 INTRODUCTION	1
SECTION 2 CONCLUSIONS	2
SECTION 3 RECOMMENDATIONS	8
SECTION 4 DESCRIPTION OF THE PROBLEM	10
COMBINED SEWER OVERFLOW TREATMENT REQUIREMENTS AT FORT WAYNE	10
SELECTION OF SITE	11
DESCRIPTION OF THE COMBINED SEWER SERVICE AREA	11
DESCRIPTION OF THE STORMWATER PUMPING AND SCREENING FACILITY	15
SECTION 5 EXPERIMENTAL PROGRAM	24
PRE-CONSTRUCTION EVALUATION PHASE	24
River Sampling Locations	24
Pre-Construction Sampling Program	25
POST-CONSTRUCTION EVALUATION PHASE	26
Screening Equipment Locations and Flow Measurements	26
Post-Construction Sampling Program	31
Screening Equipment Efficiency Evaluations	35
Stormwater Screening Facility Impact on Water Pollution Control Plant	36
Determination of Shock and Total Pollutional Loads on the Receiving Stream	37
Screening Methods Cost Analysis	37

TABLE OF CONTENTS (CONTINUED)

PAGE

SECTION 6 RESULTS OF THE EXPERIMENTAL PROGRAM 39

- STREAM WATER QUALITY IMPROVEMENTS 39
  - Introduction 39
  - Dissolved Oxygen 39
  - Biochemical Oxygen Demand 46
  - Fecal Coliform Group Indicator Organisms 47
  - Total Phosphorus 49
  - Ammonia Nitrogen 53
  - Suspended Solids (Nonfilterable Residue) 53
  - Stormwater Facility Evaluation Parameters 54
    - Settleable and Flotable Debris 54
    - Hydraulic Capacity 57
  - Stormwater Screening Equipment Comparative Evaluation 62
    - Suspended Solids (Nonfilterable Residue) 62
    - Biochemical Oxygen Demand (BOD_5) 71
    - Total Phosphorus (Total P) 76
    - Ammonia Nitrogen (NH_4^+-N) 80
    - Fecal Coliform Group Indicator Bacteria 81
    - Screening Efficiency Net Overall Treatment Efficiency 81
  - Operations, Maintenance and Special Considerations 81
    - Stormwater Facility Conduits and Wetwell 81
    - Stormwater Pumps 82
    - Stormwater Distribution System 82
    - Bauer Hydrasieve® 82
    - Rex Rotary Drum Screen 83
    - SWECO CWC® Centrifugal Wastewater Concentrator 86
    - Stormwater Facility Chlorination 89
    - Effectiveness of the Stormwater Facility in Dealing with Combined Sewer Overflows 91
  - Stormwater Effects on Terminal Pond 92
    - Dissolved Oxygen 99
    - Biochemical Oxygen Demand 100
    - Chemical Oxygen Demand 107
    - Suspended Solids (Nonfilterable Residue) 111
    - Total Phosphorus 116

SECTION 7 COST ANALYSIS FOR THE PROGRAM 118

- CONSTRUCTION COSTS FOR THE STORMWATER FACILITY 118
- ANNUAL COSTS OF OPERATION AND MAINTENANCE 121

REFERENCES 124

LIST OF FIGURES

FIGURE		PAGE
1	AERIAL PHOTOGRAPH OF WATER POLLUTION CONTROL PLANT - CITY OF FORT WAYNE, INDIANA	12
2	AERIAL PHOTOGRAPH OF STORMWATER TREATMENT FACILITY - CITY OF FORT WAYNE, INDIANA	12
3	SERVICE AREA LOCATION PLAN - CITY OF FORT WAYNE, INDIANA	13
4	WATER POLLUTION CONTROL PLANT - 84-INCH BYPASS DIVERSION CHAMBER	14
5	WATER POLLUTION CONTROL PLANT - STORMWATER TREATMENT FACILITIES	16
6	STORMWATER TREATMENT FACILITY - CITY OF FORT WAYNE, INDIANA	17
7	CONSTANT HEAD TANK RAW STORMWATER DISTRIBUTION DEVICE	17
8	STORMWATER TREATMENT FACILITY FLOW ROUTING SCHEMATIC DIAGRAM	18
9	BAUER HYDRASIEVE® AS ORIGINALLY INSTALLED	19
10	BAUER HYDRASIEVE® AS MODIFIED WITH SPLASH GUARDS - EAST BAUER	19
11	REX ROTARY DRUM SCREEN	21
12	REX ROTARY DRUM SCREEN AUTOMATIC BACKWASH COLLECTION TROUGH	21
13	SWECO CWC-60® CENTRIFUGAL WASTEWATER CONCENTRATOR	23
14	NORTH BANK OF SWECO CWC-60® SCREENING UNITS	23
15	MAUMEE RIVER MEAN STREAM DISSOLVED OXYGEN - ANTHONY BOULEVARD BRIDGE - JANUARY, 1970 TO FEBRUARY, 1976	43
16	MAUMEE RIVER MEAN STREAM DISSOLVED OXYGEN - U.S. 30 BYPASS BRIDGE - JANUARY, 1970 TO FEBRUARY, 1976	44

LIST OF FIGURES (CONTINUED)

FIGURE		PAGE
17	MAUMEE RIVER MEAN STREAM DISSOLVED OXYGEN - NEW HAVEN BRIDGE AT LANDIN ROAD - JANUARY, 1970 TO FEBRUARY, 1976	45
18	STREAM FECAL COLIFORM GROUP INDICATOR ORGANISMS - PRE-CONSTRUCTION EVALUATION PHASE - AUGUST, 1971 TO NOVEMBER, 1972	50
19	STREAM FECAL COLIFORM GROUP INDICATOR ORGANISMS - POST-CONSTRUCTION EVALUATION PHASE - JANUARY, 1975 TO FEBRUARY, 1976	51
20	SCREENED MATERIAL ADHERING TO TRASH RACK VERTICAL BARS	56
21	INTERIOR VIEW OF STORMWATER FACILITY TRASH RACK SHOWING STRINGY DEBRIS	56
22	JOHNSTON 30 PS PUMP COLUMN AND SUCTION BELL AFTER ELEVEN MONTHS OF OPERATION	56
23	CHLORINE CONTACT CHANNEL FLOTABLE DEBRIS COLLECTED AT SCUM REMOVAL PIPE - JULY 24, 1975	58
24	BAUER HYDRASIEVE® SCREENINGS AFTER STORMWATER EVENT	58
25	CLOSE-UP OF SEMIDRY SCREENINGS COLLECTED DURING STORMWATER EVENT FROM BAUER HYDRASIEVE®	58
26	BAUER HYDRASIEVE® CENTER CONCENTRATE CHANNEL SHOWING SEMIDRY SCREENINGS COLLECTED	59
27	BAUER HYDRASIEVE® OPERATING AT FULL HYDRAULIC CAPACITY	59
28	REX ROTARY DRUM SCREEN HEADBOX OVERFLOW POINT	61
29	CHLORINE CONTACT CHANNEL FROM OVERFLOW WEIR TO TERMINAL POND SHOWING SLUDGE BANK DEPOSITS	61
30	CHLORINE CONTACT CHANNEL BOTTOM AT OVERFLOW WEIR TO TERMINAL POND	61
31	SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR SUSPENDED SOLIDS (NONFILTERABLE RESIDUE) AS A FUNCTION OF HYDRAULIC LOADING RATE	68

LIST OF FIGURES (CONTINUED)

FIGURE		PAGE
32	SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR SUSPENDED SOLIDS (NONFILTERABLE RESIDUE) AS A FUNCTION OF SOLIDS LOADING RATE	70
33	SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR BIOCHEMICAL OXYGEN DEMAND (BOD$_5$) AS A FUNCTION OF HYDRAULIC LOADING RATE	72
34	SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR BIOCHEMICAL OXYGEN DEMAND (BOD$_5$) LOADING RATE	73
35	SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR CHEMICAL OXYGEN DEMAND (COD) AS A FUNCTION OF HYDRAULIC LOADING RATE	75
36	SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR CHEMICAL OXYGEN DEMAND (COD) AS A FUNCTION OF COD LOADING RATE	77
37	SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR TOTAL PHOSPHORUS (TOTAL P) AS A FUNCTION OF HYDRAULIC LOADING RATE	78
38	SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR TOTAL PHOSPHORUS (TOTAL P) AS A FUNCTION OF TOTAL P LOADING RATE	79
39	REX ROTARY DRUM SCREEN OPERATING ON RECIRCULATED POND WATER	85
40	SWECO CWC® SCREENS IN OPERATION ON RAW STORMWATER OVERFLOWS SHOWING EFFLUENT SUDSING EFFECT	90
41	SWECO INFLUENT DISTRIBUTION MANIFOLD FOR SOUTH BANK OF CWC UNITS	90
42	STORMWATER TREATMENT FACILITY FLOW ROUTING SCHEMATIC DIAGRAM	93
43	RAINFALL HYDROGRAPH AND LONGITUDINAL HYDRAULIC FLOW IN MGD - PART 1 AND PART 2	95
44	LONG-TERM PLANT RAW SEWAGE PUMPING DAILY HYDRAULIC FLOW IN MGD	98
45	WATER POLLUTION CONTROL PLANT EFFLUENT DISSOLVED OXYGEN LONG-TERM MEAN MONTHLY D.O. CONCENTRATION	101

LIST OF FIGURES (CONTINUED)

FIGURE		PAGE
46	PLANT TERMINAL POND EFFLUENT BIOCHEMICAL OXYGEN DEMAND	103
47	WATER POLLUTION CONTROL PLANT EFFLUENT BIOCHEMICAL OXYGEN DEMAND (BOD_5) LONG-TERM MEAN MONTHLY BOD_5 CONCENTRATION	104
48	LONG-TERM TERMINAL POND MEAN MONTHLY ORGANIC LOADING RATE	108
49	TERMINAL POND MEAN MONTHLY BOD_5 LOADING REMOVAL EFFICIENCY AS A FUNCTION OF POND INFLUENT BOD_5 LOADING RATE	109
50	TERMINAL POND EFFLUENT CHEMICAL OXYGEN DEMAND LONG-TERM MEAN MONTHLY CONCENTRATION	110
51	WATER POLLUTION CONTROL PLANT EFFLUENT SUSPENDED SOLIDS LONG-TERM MEAN MONTHLY SUSPENDED SOLIDS CONCENTRATION	112
52	LONG-TERM TERMINAL POND MEAN MONTHLY SUSPENDED SOLIDS LOADING RATE	114
53	TERMINAL POND MEAN MONTHLY SUSPENDED SOLIDS LOADING REMOVAL EFFICIENCY AS A FUNCTION OF POND INFLUENT LOADING RATE	115
54	WATER POLLUTION CONTROL PLANT EFFLUENT TOTAL PHOSPHORUS LONG-TERM MEAN MONTHLY CONCENTRATION	117

LIST OF TABLES

TABLE		PAGE
1	PRE-CONSTRUCTION EVALUATION PHASE SAMPLING PLAN - AUGUST, 1971 TO NOVEMBER, 1972	27
2	POST-CONSTRUCTION EVALUATION PHASE SAMPLING PLAN - JANUARY, 1975 TO FEBRUARY, 1976	32
3	PRE-CONSTRUCTION EVALUATION PHASE STREAM DISSOLVED OXYGEN - AUGUST, 1971 TO NOVEMBER, 1972	41
4	POST-CONSTRUCTION EVALUATION PHASE STREAM DISSOLVED OXYGEN - JANUARY, 1975 TO FEBRUARY, 1976	41
5	NUMBER OF DAYS STREAM DISSOLVED OXYGEN LESS THAN 4.0 mg/l	42
6	PRE-CONSTRUCTION EVALUATION PHASE STREAM BIOCHEMICAL OXYGEN DEMAND - AUGUST, 1971 TO NOVEMBER, 1972	46
7	POST-CONSTRUCTION EVALUATION PHASE STREAM BIOCHEMICAL OXYGEN DEMAND - JANUARY, 1975 TO FEBRUARY, 1976	47
8	PRE-CONSTRUCTION EVALUATION PHASE STREAM FECAL COLIFORM GROUP INDICATOR ORGANISMS - AUGUST, 1971 TO NOVEMBER, 1972	48
9	POST-CONSTRUCTION EVALUATION PHASE STREAM FECAL COLIFORM GROUP INDICATOR ORGANISMS - JANUARY, 1975 TO FEBRUARY, 1976	49
10	PRE-CONSTRUCTION EVALUATION PHASE STREAM TOTAL PHOSPHORUS - AUGUST, 1971 TO NOVEMBER, 1972	49
11	POST-CONSTRUCTION EVALUATION PHASE STREAM TOTAL PHOSPHORUS - JANUARY, 1975 TO FEBRUARY, 1976	52
12	POST-CONSTRUCTION EVALUATION PHASE AMMONIA NITROGEN - JANUARY, 1975 TO FEBRUARY, 1976	53
13	STREAM SUSPENDED SOLIDS (NONFILTERABLE RESIDUE)	54
14	STORMWATER FACILITY WETWELL SEDIMENTS SAMPLING SURVEY RESULTS - DECEMBER 8, 1975	55
15	SCREENING EQUIPMENT COMPARATIVE EVALUATION OVERALL TREATMENT EFFICIENCY	63

LIST OF TABLES (CONTINUED)

TABLE		PAGE
16	SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR SUSPENDED SOLIDS (NONFILTERABLE RESIDUE)	69
17	SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR VOLATILE SUSPENDED SOLIDS (VSS)	71
18	SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR BIOCHEMICAL OXYGEN DEMAND (BOD_5)	71
19	SCREENING FACILITY FILTERABLE BOD_5 DETERMINATIONS - EVENT NO. 207	74
20	SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR CHEMICAL OXYGEN DEMAND	76
21	SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR TOTAL PHOSPHORUS (TOTAL P)	80
22	SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR AMMONIA NITROGEN	80
23	SCREENING EQUIPMENT NET OVERALL TREATMENT EFFICIENCY IN PERCENT	81
24	WATER POLLUTION CONTROL PLANT TERMINAL POND EFFLUENT DISSOLVED OXYGEN	99
25	PLANT TERMINAL POND EFFLUENT BIOCHEMICAL OXYGEN DEMAND	102
26	PLANT SECONDARY TREATMENT EFFLUENT BIOCHEMICAL OXYGEN DEMAND	105
27	PLANT TERMINAL POND EFFLUENT SUSPENDED SOLIDS (NONFILTERABLE RESIDUE)	111
28	PLANT SECONDARY TREATMENT EFFLUENT SUSPENDED SOLIDS	113
29	DETAILED COSTS OF CONSTRUCTION OF FORT WAYNE STORMWATER TREATMENT FACILITY	118
30	COMPARATIVE ANNUAL OPERATIONS AND MAINTENANCE COSTS FOR THREE SCREENING METHODS	123

ACKNOWLEDGMENTS

No project of this scope and magnitude would be possible without the assistance of many dedicated people who have taken an interest in the conduct of the work. We wish to extend our sincere appreciation and acknowledgment to the following individuals who have been connected with this project throughout its life:

Mr. Ron Bonar, Consultant and Former City Engineer
City of Fort Wayne

Mr. Phil Boller, Pollution Control Engineer
City of Fort Wayne

Mr. Dick Koos, Water Pollution Control Plant Chemist
City of Fort Wayne

The following individuals at Howard Needles Tammen & Bergendoff, Henry B. Steeg & Associates Division, deserve special credit for their interest and cooperation during this project:

Mr. Jerry Lynch, Project Representative
Mr. Charles F. Niles, Jr., Special Project Consultant
Mr. George K. Erganian, Partner

The Contractor for the Stormwater Facility was:

Bowen Engineering Company, Inc.
Indianapolis, Indiana

The following companies are recognized for their continuing interest and assistance throughout this project:

Bauer Bros. Division of Combustion Engineering, Inc.
Envirex, Inc., Water Quality Control Division
SWECO, Inc., Environmental Systems Division

Special thanks to Byron L. Anderson of B. L. Anderson, Inc., for his technical assistance and special interest.

SECTION 1

INTRODUCTION

One of the major difficulties confronting most urbanized areas in this country is the so-called "stormwater" problem. Every community in this nation faces this problem at some time or another. The difficulty with dealing with this problem is its quantity and quality variations. However, the manner of dealing with storm-caused pollutant-laden wastewater has historically been a temporary, local solution. The net effect of these projects designed to "solve" a local stormwater-caused problem has frequently resulted in a transfer of the problem of storm-caused water pollution to a later time frame and greater distance from the source.

So it is with one component of this stormwater problem that has been termed "combined sewer overflows". The sewers were designed to carry dry-weather sanitary wastes in the same conduits as wet-weather stormwater generated by precipitation over and above the dry-weather flows. This storm-caused combined sewer flow usually is greater than the capability of the treatment plant to handle hydraulically, let alone treat to the equivalent of secondary treatment removal efficiencies.

This work was intended to determine the effectiveness and cost of screening combined sewer overflows from plant bypass wastewaters by each of the screening methods consisting of side hill vertical fixed screens, centrifugal wastewater concentration and rotary drum screenings followed by two-day terminal ponding of screened effluent plus secondary treatment plant effluent to evaluate the benefits to the receiving waters by the utilization of the methods cited for capturing and treating plant bypass of combined sewer overflows, and to quantify the effectiveness of the screening methods studied as tertiary treatment units on the two-day terminal ponds.

SECTION 2

CONCLUSIONS

1. There has been an increase in stream dissolved oxygen since the diversion of the combined sewer overflows to the Stormwater Treatment Facility.

   a. The relative quantity of raw, untreated combined sewer overflows originating from storm-caused bypassing at the Water Pollution Control Plant has been reduced by 28 percent.

   b. The mean stream dissolved oxygen observed showed a measurable increase since the start-up of the Stormwater Facility to an average concentration of 8.9 milligrams per liter (mg/l), from pre-construction mean of 7.4 mg/l, at a point 7-1/4 kilometers downstream.

   c. The frequency of stream dissolved oxygen levels less than 4.0 mg/l measured at a point 7-1/4 kilometers downstream decreased from 23 days per year to 11 days per year since construction of the Stormwater Facility. This dissolved oxygen improvement is not solely attributed to the Stormwater Facility since upstream dissolved oxygen levels less than 4.0 mg/l also decreased from 6.9 days per year to 0.04 days per year. These results are statistically significant at the 95 percent confidence level.

2. There has been a decrease in the stream biochemical oxygen demand since the construction and operation of the Stormwater Facility.

   a. An estimated 54 percent reduction in the overflow quantity of BOD_5 has been achieved since the start-up of the Stormwater Facility.

   b. The range of values of stream BOD_5 has been decreased since the start-up of the Stormwater Facility - from 0.5 to 25 mg/l in the pre-construction phase to 0.2 to 11.0 mg/l in the post-construction phase. However, the upstream BOD_5 experienced a similar reduction.

3. There has been a significant improvement in the stream water quality as measured by membrane filter method for fecal coliform group indicator bacteria.

   a. The stream geometric mean fecal coliform was 9,000 organisms per 100 milliliters in the pre-construction phase and 1,000 per 100 milliliters in the post-construction phase.

   b. There must be measurable water quality improvements for fecal coliform upstream from the Water Pollution Control Plant to show any additional improvements in stream fecal coliform downstream from the plant.

4. Decrease in the stream total phosphorus after completion of the Water Pollution Control plant chemical precipitation phosphorus removal and Stormwater Facility are mostly attributable to the chemical precipitation of phosphorus at the plant.

5. There was not distinguishable reduction in stream suspended solids over the evaluation period in spite of an estimated 70 percent reduction in suspended solids originating in storm-caused combined sewer overflows.

6. Among the screening methods evaluated, the fixed vertical side hill hydraulic sieve (Bauer Hydrasieve®) was shown to be most effective in the removal of coarse solids debris.

   The Bauer unit produced a semidry solid residue that could be removed and trucked off to a sanitary landfill without further treatment at a hydraulic loading rate of approximately 25 gpm per square foot (1,050 lpm/m^2).

7. Of the three screening methods studied none of the pollutant removal efficiencies were shown to be statistically different from zero at the 95 percent confidence level.

SCREENING EQUIPMENT OVERALL TREATMENT
EFFICIENCY IN PERCENT*

	Bauer Hydrosieve® 11 Mesh	Rex Rotary Drum Screen 100 Mesh	SWECO CWC® 165 Mesh
Suspended Solids	15.7 (4.7)	15.8 (9.2)	26.6 (22.6)
Biochemical Oxygen Demand	13.1 (5.7)	14.4 (3.4)	16.5 (3.1)
Chemical Oxygen Demand	13.6 (6.5)	11.3 (1.5)	19.5 (12.9)
Total Phosphorus	13.3 (6.9)	11.4 (2.0)	18.9 (8.3)

*numbers in parenthesis are the overall treatment efficiencies including negative efficiency data.

8. Negative treatment efficiencies calculated for screening units may be due to sampling, analytical and compositing errors. Excessive fluid shear may also account for a portion of the variation and negative results. The negative removal efficiencies for suspended solids, for example, are calculated to be as high as -85 percent although they generally fall between 0 and -30 percent. Actual negative removals are impossible for suspended solids since the screening units are physical barriers.

9. There does not appear to be a correlation between screening efficiencies and hydraulic loading rates for any of the screens studied. There does appear to be an event-by-event variation in treatment efficiencies independent of the hydraulic loading.

10. There was no discernable trend in screening rate efficiencies as a function of pollutant unit loading rates for any of the parameters studied. Rather, there appeared to be a wide dispersion of results from event to event independently of unit loadings.

11. None of the screening methods studied appeared to have any effect on the removal of fecal coliform indicator bacteria.

12. The fixed vertical side hill screening unit makes an excellent screening device for removal of solid debris such as wood pieces, metals, plastic materials, leaves and twigs, and other such gross solids:

    a. As a screening unit in and of itself, the Bauer unit performance is comparable to the other screens studied.

    b. As a pretreatment device, the Bauer was capable of reduction of net overall pollutant loadings to subsequent treatment methods.

    c. Only routine preventive maintenance was found to be necessary for this unit.

    d. Headloss considerations are important for the Bauer because it requires about two meters (6.6 feet) static headloss to operate.

    e. Minimum floor area requirements for this installation were 66.3 square meters or approximately 0.85 square meters per million liters per day hydraulic capacity.

    f. There were no auxiliary power or other services required for this installation.

    g. Semidry screenings were possible by operating the Bauer unit at about 3/4 of maximum hydraulic capacity.

13. Screening of combined sewer overflows by the rotary fine screening method (Rex Rotary Drum Screen) resulted in the following conclusions:

    a. The Rex rotary drum screen installed at Fort Wayne required more corrective mechanical maintenance on the part of plant personnel than any other screening method.

    b. The Rex unit experienced one screen failure in the evaluation period. The SWECO unit had 142 screen failures in the same time period.

c. This particular unit required electrical power supply for the drive motor and the backwash pump motors and controllers and potable water as make-up for backwash.

d. The Rex unit minimum floor area is 38 square meters or 0.54 square meters per million liters per day hydraulic capacity.

e. The Rex unit requires about 3/4 meter (2.4 feet) static headloss to operate on storm-caused overflows.

f. Approximately 225 liters per minute (60 gpm) screenings concentrate was generated when the automatic backwash system was operating properly.

g. It was not possible to operate this particular unit automatically at the Fort Wayne Stormwater Facility because of hydraulic loading problems.

14. The centrifugal concentration method of screening was observed to infer the following conclusions:

a. It was not possible to operate the SWECO CWC® unit automatically at the Fort Wayne Stormwater Facility because the backwash sensing circuits called for continuous backwashes, drastically reducing screen through-put.

b. Manual backwash procedures were necessary to keep the units ready for service at any time.

c. Screen failures from punctures adversely affected performance of the CWC units because a single screen failure reduced screening unit effectiveness dramatically.

d. At least 1.8 to 3.7 meters (6 to 12 feet) static hydraulic head is required to operate these units.

e. Potable water was required at Fort Wayne to insure that there was a clean source of make-up water available.

f. These units require electrical power, potable water, pneumatic air, and during winter months environmental control (heating) to be operational.

g. Four of these units required 48 square meters or 0.35 square meter per million liters per day hydraulic capacity.

h. These units result in screenings concentrate from 10 to 20 percent of raw flow to the unit.

15. There were differences among the screening methods studied in quality and quantity of concentrated screenings generated. However, from the data obtained in this program, there were no statistical differences in quality of the combined screenings facility concentrate when compared to raw wastewater.

16. There were no noticeable effects of prechlorination of pond influent wastewaters although those data obtained indicate a substantial reduction in fecal coliform bacteria. Only limited data are available on pond flora and fauna because of time and budget constraints.

17. Chlorination rate for storm-caused overflows at the Fort Wayne Stormwater Facility ranged from 6.0 kilograms chlorine per million liters (50 pounds per MG) to 21.1 kilograms per million liters (176 pounds per MG) and a mean rate of 13.3 kilograms per million liters (111 pounds per MG). This resulted in a dosage of 6.0 to 21.1 mg/l.

18. The terminal pond, receiving flows from both the Water Pollution Control Plant and the Stormwater Facility, was shown to be capable of absorbing a 100 percent increase in flow, dampen the peak flow by its available volume, and allow this peak flow to pass through the teatment system much more gradually than it might had the Stormwater Facility never been constructed.

    a. Furthermore, it gave the storm-caused overflows at the minimum screening plus chlorination plus mixing with plant treated effluent and detention in the terminal ponds.

    b. There was no evidence of pond treatment short-circuiting.

    c. Plant operators became much more conscious of plant bypasses as a consequence of having to deal with the overflows at the Stormwater Facility.

19. The terminal pond treatment efficiency for the removal of stormwater overflow pollutants was noted:

    a. No perceptible effect of the addition of stormwater overflows could be detected for pond effluent dissolved oxygen, fecal coliform, total phosphorus, or ammonia nitrogen.

    b. Pond effluent biochemical oxygen demand data show decreased monthly mean terminal pond effluent concentrations attributed to the addition of storm-caused overflows for the post-construction period as compared to preceding years.

    c. Algal blooms and negative treatment efficiencies for BOD_5 were observed with the lower pond loading rates (i.e., less than 112 kg BOD_5/hectare/day or 100 pounds BOD_5/acre/day). In contrast to this, more stable, higher efficiencies of removal for BOD_5 were observed with the higher pond loading rates (more than 224 kg

BOD_5/hectare/day or 200 pounds BOD_5/acre/day). Average removal efficiency for the pond effluent BOD_5 in the post-construction period was 75 percent ($\sigma = \pm 4.8$ percent).

d. The addition of storm-caused overflows resulted in shorter hydraulic detention time and higher organic loadings which intended to inhibit algal growth in the warm weather months.

e. Short method chemical oxygen demand data did not follow the same trend of values as BOD_5. No data regarding reflux method COD are available.

f. Terminal pond effluent loadings for suspended solids were shown to be a very heavily damped response to the pond influent loadings.

g. Pond effluent suspended solids concentrations correspond closely to comparable BOD_5 concentrations.

h. Average mean monthly removal efficiency for suspended solids was 74.6 percent ($\sigma = \pm 14$ percent).

20. The comparative annual operations and maintenance costs per million gallons of treated flow for the three screening methods were:

a. Bauer Hydrasieve®, $25.25/MG

b. Rex Rotary Drum Screen, $50.00/MG

c. SWECO CWR® Centrifugal Wastewater Concentrator, $59.68/MG

SECTION 3

RECOMMENDATIONS

1. Additional research into the nature of pollutant removals in stormwater detention ponds with short holding times is needed. Specifically, more research is needed to develop sound engineering design loading parameter ranges for single purpose and dual-use terminal ponds intended to treat stormwater and storm-caused combined sewer overflows.

2. Some provisions for sludge removal should be built in all piping, conduits, wetwells, channels and ponds designed to handle combined sewer overflows to remove the build-up of sediments from these wastewaters.

3. Consideration of flow control to each individual screening unit is essential to maintaining maximum treatment efficiencies. Specifically, the Bauer Hydrasieve® should have some way to differentially distribute the flows to be treated to each module in order to optimize the hydraulic flux rate to each screen module.

4. Some provision for direct collection and disposal of the screenings from the Bauer Hydrasieve® should be considered. These screenings can be directly collected semidry from the screen surface for ultimate disposal.

5. Some thought on the problems associated with the routine preventive maintenance should be given for all components used in a combined sewer treatment facility. Automatic oilers for all electrical motors and bearings are a must. Intermittent service on storm-caused combined sewer overflows requires sturdy, easily cleaned, checked, lubricated and maintained components.

6. Some type of prescreening ahead of fine screens is necessary to prevent premature failure of the screen cloths from stormwater-borne flotsam and debris. The Bauer Hydrasieve® performed very well in this role.

7. Automatic controls for screens treating combined sewer overflows and installed in remote facilities should be monitored using hard-wired remote devices or real-time telemetry to a central operator control station continuously manned year-round. Periodic equipment checks by service and control operating personnel are necessary to maintain maximum treatment efficiency of the units.

8. Experimentation with heavier gage screens or tougher, more durable materials in the upper cage screens for the rotary centrifugal screens is recommended to improve the screening efficiency and screen life of these units.

SECTION 4

DESCRIPTION OF THE PROBLEM

COMBINED SEWER OVERFLOW TREATMENT REQUIREMENTS AT FORT WAYNE

The City of Fort Wayne is located at the junction of the Maumee, St. Joseph and St. Marys Rivers in northeastern Indiana. The Maumee River flows northeasterly from Fort Wayne and eventually enters Lake Erie at Toledo, Ohio. A report by the U.S. Federal Water Quality Administration in 1968, "The Lake Erie Report"[1], identified the extensive combined sewer overflow system in the communities on the upper reaches of the Maumee as a major source of highly polluted raw wastewater. A major part of the lake pollution cited in this report was attributed to the Maumee River loadings.

The City of Fort Wayne authorized a study of the entire collection and treatment system of the City in 1969. The thrust of this study was to determine the pollution sources, quantify them, and outline a Master Plan for correction of the major sources of pollutants.[3] Within the scope of this city-wide Master Plan was the investigation of the entire existing sanitary and stormwater collection system, including combined sewer overflows.

As a result of the Master Plan, the urban drainage areas of some 12,000 acres were found to have 150 separate trunk sewer systems. Approximately three-fourths were found to be combined sanitary and storm sewers. These combined service trunk sewers were interlocked throughout with more than thirty diversions and regulators, resulting in more than twenty separate overflow points.

This Master Plan identified the largest sources of discharges by capacity and pollution potential. A three-phase plan for combined sewer overflow detention of stormwater-induced hydraulic surges was recommended. A key feature of the plan was the elimination of some combined sewer overflow points by construction of a centrally-located stormwater treatment facility (identified as Facility "A" in the Master Plan)[3] near the Water Pollution Control Plant. This facility was to be designed to handle all of the excess hydraulic flow received at the Water Pollution Control Plant, apply the equivalent of primary treatment to remove the flotables and settleables, and disinfect the entire flow to reduce the pathogenic organisms before discharge to the Maumee.

SELECTION OF SITE

City Officials had selected a site across the river from the Water Pollution Control Plant to construct a dual-use two-day terminal pond and chlorination facility for disinfection and polishing of the Water Pollution Control Plant flow (completed in 1971). This site was readily available for construction of the proposed Facility "A" to be dedicated exclusively for treatment of combined sewer stormwater overflows. As originally proposed in the Master Plan, a large pumping station for combined sewer overflows was to be built discharging raw stormwater-induced overflows at sufficient hydraulic static lift to flow into conventional primary sedimentation basins.

However, the magnitude of the work and construction costs of full scale installations elsewhere encouraged community interest in conducting a full scale demonstration grant program at Fort Wayne using screening as an alternate to primary sedimentation. An application for a Research, Demonstration and Development Grant for evaluation of this alternate was submitted to the U.S. Environmental Protection Agency in October, 1970. After several additions and revisions suggested in response to developments elsewhere, a program plan of study was approved using several methods of fine mesh screening, replacing the original concept of conventional primary sedimentation altogether.

The City of Fort Wayne Water Pollution Control Plant on the City's east side at Dwenger Avenue was scheduled for plant expansion to start in 1975 (shown under construction in this recent aerial photograph, Figure 1). The 36-acre body of water shown in the upper left-hand corner of Figure 1 is the dual-use terminal pond described above. The new construction in the photograph shown on the north side of the river is to become new Terminal Pond No. 2, another dual-use pond.

Figure 2 is an aerial photograph of the Stormwater Treatment Facility showing the entire terminal pond, the Pumping and Screening Building, the chlorine contact channel, the pump wetwell overflow channel, and the terminal pond final effluent channel.

DESCRIPTION OF THE COMBINED SEWER SERVICE AREA

Figure 3 is a map of the location of the Water Pollution Control Plant, the Stormwater Facility, and other important service area landmarks and locations.

The area served by the stormwater facilities in this program included approximately eight miles of combined trunk sewers in a drainage area of 995 acres on the south side of the river. However, raw sewage flow could be diverted from the Water Pollution Control Plant influent headworks at any time by simply allowing the head to build up in the plant interceptors until the overflow elevation was reached. See Figure 4 for a section drawing of this diversion structure in the vicinity of the headworks of the Water Pollution Control Plant.

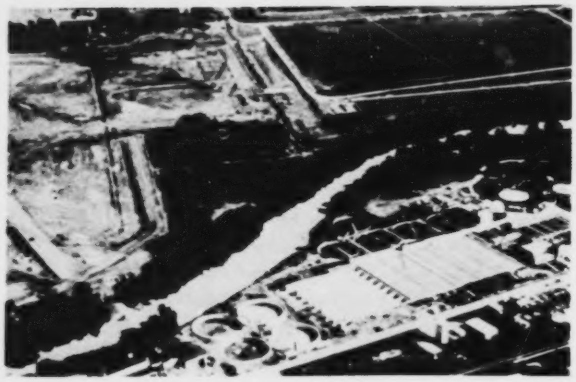


Figure 1 - Aerial Photograph of
Water Pollution Control Plant
City of Fort Wayne, Indiana

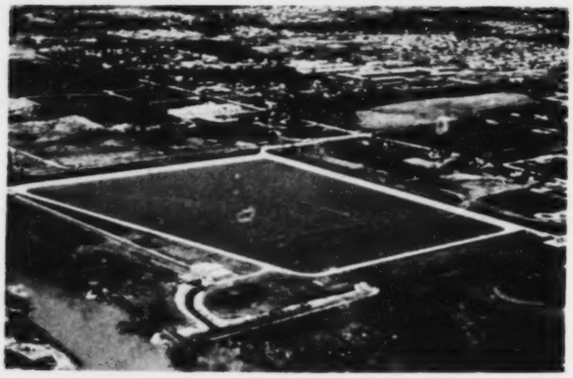


Figure 2 - Aerial Photograph of Stormwater
Treatment Facility
City of Fort Wayne, Indiana

Reproduced from
best available copy.

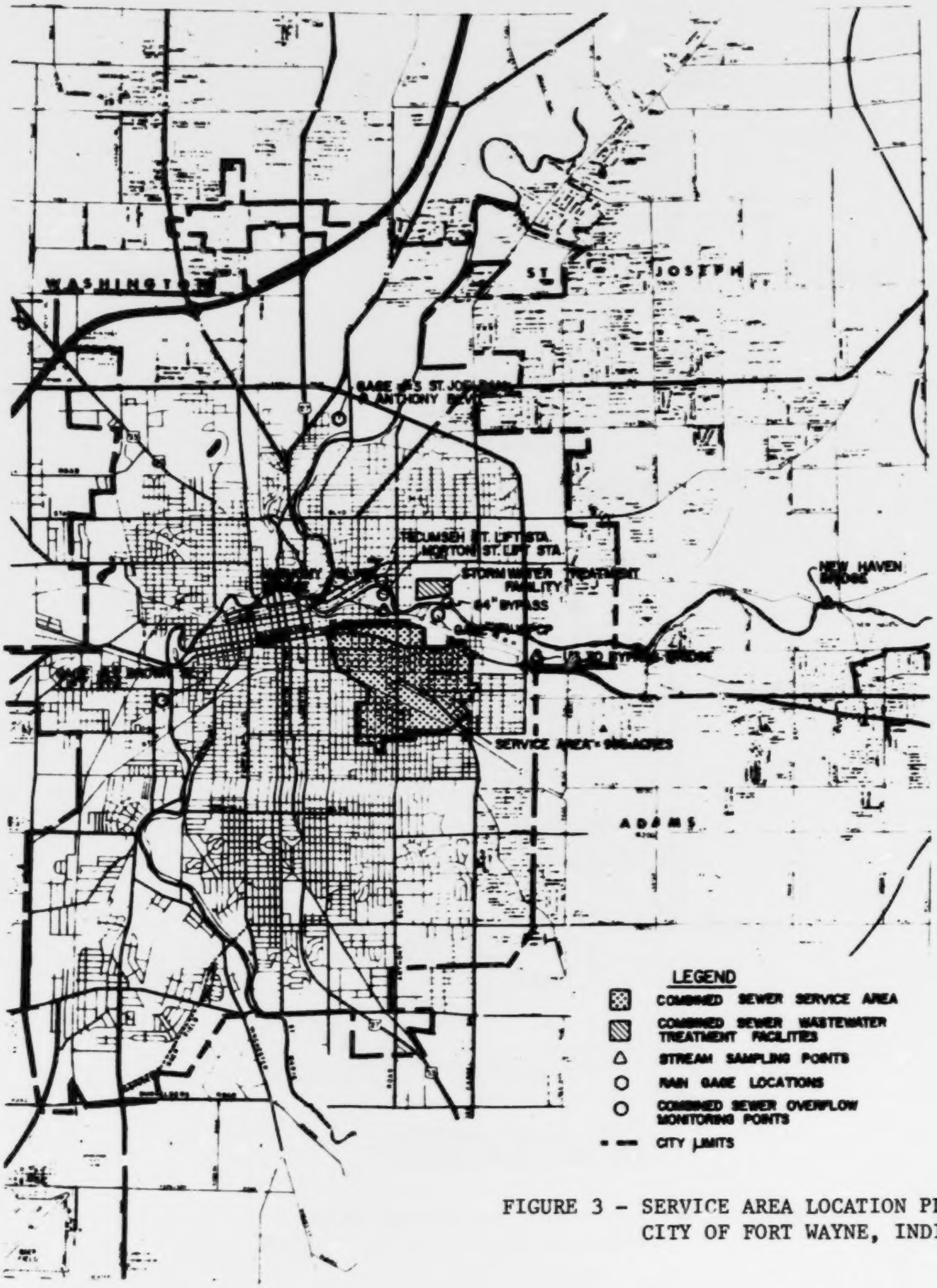
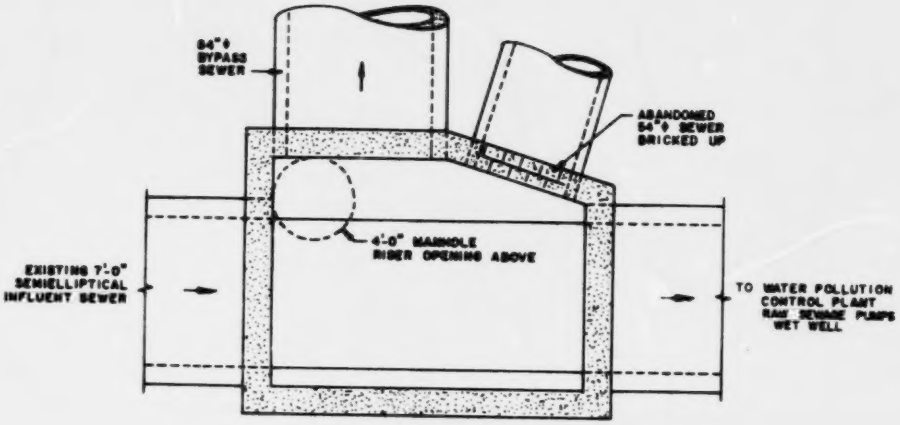
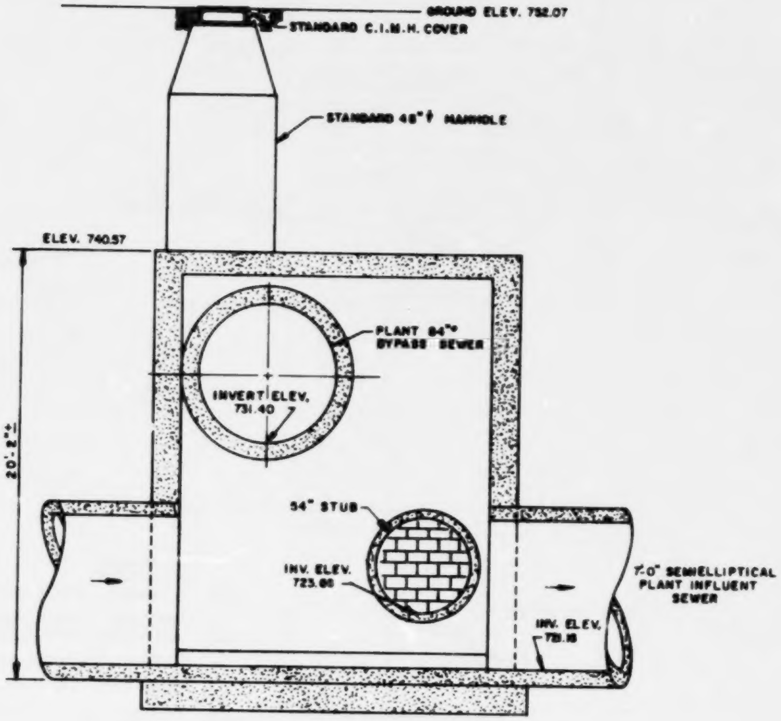

WASHINGTON
ST JOSEPH
ADAMS
GAGE AT ST. JOSEPH AND ST. ANTHONY BLVD.
TECUMSEH ST. LIFT STA.
MORTON ST. LIFT STA.
STORM WATER TREATMENT FACILITY
84" BYPASS
NEW HAVEN BRIDGE
SERVICE AREA
LEGEND
COMBINED SEWER SERVICE AREA
COMBINED SEWER WASTEWATER TREATMENT FACILITIES
STREAM SAMPLING POINTS
RAIN GAGE LOCATIONS
COMBINED SEWER OVERFLOW MONITORING POINTS
CITY LIMITS

FIGURE 3 - SERVICE AREA LOCATION PLAN
CITY OF FORT WAYNE, INDIANA


84"ϕ BYPASS SEWER
ABANDONED 54"ϕ SEWER BRICKED UP
4'-0" MANHOLE RISER OPENING ABOVE
EXISTING 7'-0" SEMIELLIPTICAL INFLUENT SEWER
TO WATER POLLUTION CONTROL PLANT RAW SEWAGE PUMPS WET WELL

PLANT BYPASS CHAMBER
PLAN VIEW


GROUND ELEV. 752.07
STANDARD C.I.M.H. COVER
STANDARD 48"ϕ MANHOLE
ELEV. 740.57
PLANT 84"ϕ BYPASS SEWER
INVERT ELEV. 731.40
20'-2"±
54" STUB
INV. ELEV. 723.88
7'-0" SEMIELLIPTICAL PLANT INFLUENT SEWER
INV. ELEV. 721.18

PLANT BYPASS CHAMBER
SECTION LOOKING NORTH

FIGURE 4 - WATER POLLUTION CONTROL PLANT
84-INCH BYPASS DIVERSION CHAMBER

As the raw wastewater builds up in the plant bypass chamber, the diverted raw wastewater flows by gravity head to a river crossing structure connected to twin 96-inch diameter conduits feeding the raw sewage wetwell at the Stormwater Pumping Facility. Figure 5 shows the location of the Water Pollution Control Plant, the connecting sewers, the WPCP chlorination facilities, Terminal Pond No. 1, and the Stormwater Treatment Facilities. From the 84-inch plant bypass chamber, the raw wastewater flows to the twin 96-inch river crossing structure. Also entering this river crossing structure is a 108-inch stormwater relief sewer constructed as a part of this project carrying the combined sewer overflows from the Glasgow Street interceptor sewer. From the twin 96-inch river crossing structure on the south side of the river, the wastewater flows to the wetwell of the stormwater pumping station.

DESCRIPTION OF THE STORMWATER PUMPING AND SCREENING FACILITY

The wetwell at the Stormwater Treatment Facility is sized at 550,000 gallons (2,082 cu. m.) capacity. A mechanically cleaned trash rake bar rack is installed at the inlet end of the wetwell. A permanent metal building houses the raw stormwater pumps and the screening equipment.

Figure 6 shows the wetwell end of the Stormwater Facility Pumping and Screening Building. The trash rake mechanism is shown in the upper position in this photograph.

Two Johnston Pump Company mixed flow vertical turbine raw stormwater pumps provide the static lift needed to bring the wetwell contents up to the treatment level. One pump, a Johnston Model 30 PS delivers 35,000 gpm at 55 feet TDH, or 50 MGD [2.191 cubic meters per second (cumec)]. The Johnston Model 24 PS pump delivers 17,500 gpm at 55 feet TDH, or 25 MGD (1.096 cumec). Each pump has its own separate suction column and discharge line with flap valve. Both discharge to a common 14-foot diameter cylindrical head tank which provides the constant head necessary to operate the screening equipment.

Figure 7 is a photograph of the stormwater pump discharge constant head tank and flow distribution device. Not shown in this photograph is the six-foot circular overflow weir inside the head tank which carries the raw stormwater overflow to the screening room effluent channel.

Figure 8 is a treatment flow routing schematic diagram showing the hydraulic flow routing in the facility.

There are three types of screening equipment installed in the screening equipment room. A side hill screen method, the Bauer Hydrasieve®, is installed in a pair of six-unit modules. The combined hydraulic capacity of these modules is normally 3,500 to 10,400 gallons per minute (0.219 cumec to 0.651 cumec) with a peak loading of 13,825 gpm (0.865 cumec).

Figure 9 is a photograph of the east bank of the Bauer units taken shortly before the facility became fully operational as a stormwater screening facility. Figure 10 is a photograph of the same screening devices showing

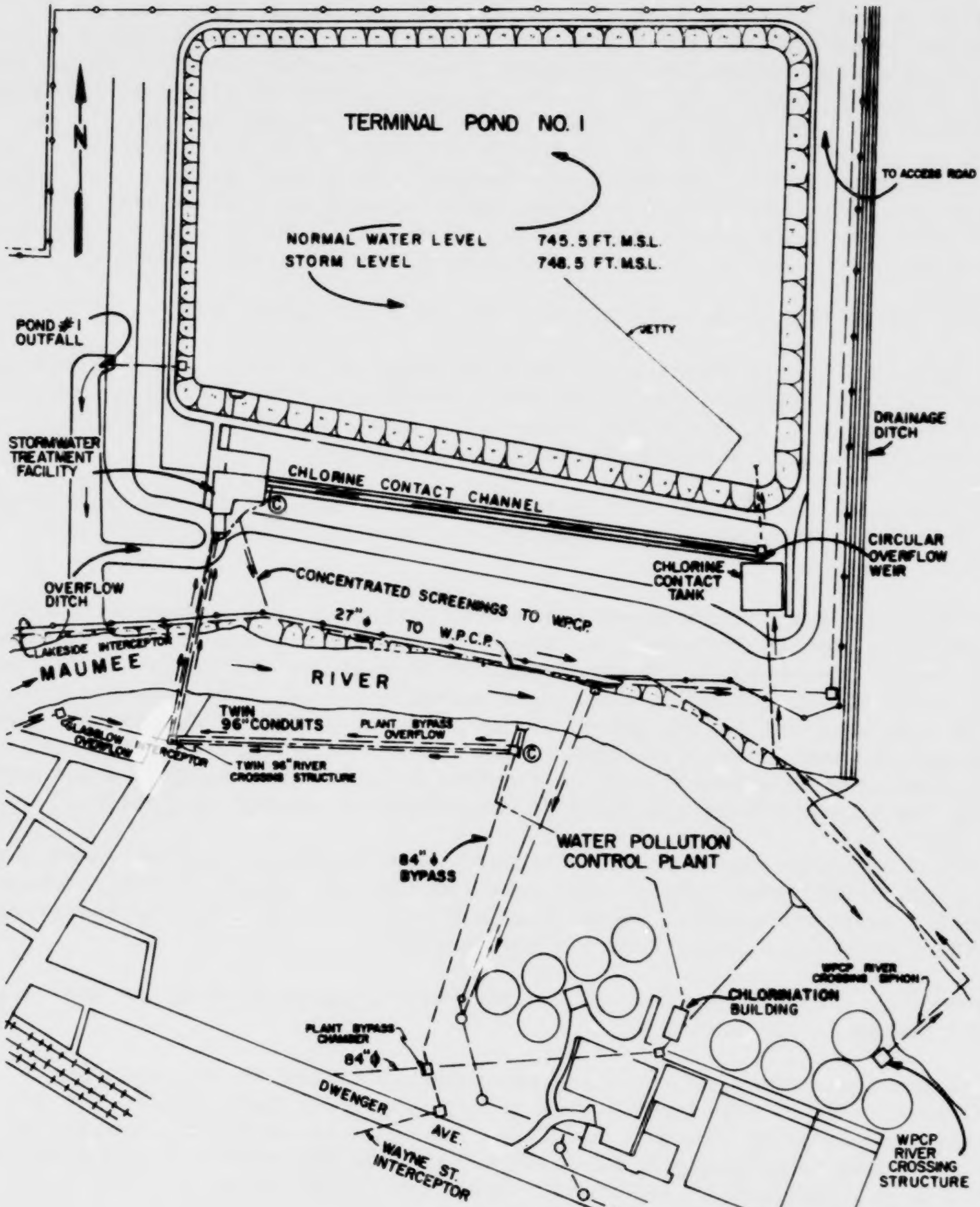

N
TERMINAL POND NO. I
TO ACCESS ROAD
NORMAL WATER LEVEL 745.5 FT. M.S.L.
STORM LEVEL 748.5 FT. M.S.L.
JETTY
POND #1 OUTFALL
DRAINAGE DITCH
STORMWATER TREATMENT FACILITY
CHLORINE CONTACT CHANNEL
CIRCULAR OVERFLOW WEIR
CHLORINE CONTACT TANK
CONCENTRATED SCREENINGS TO WPCP
OVERFLOW DITCH
27" ϕ TO W.P.C.P.
LAKESIDE INTERCEPTOR
MAUMEE
RIVER
TWIN 96" CONDUITS
PLANT BYPASS OVERFLOW
GLASSBLOW OVERFLOW
INTERCEPTOR
TWIN 96" RIVER CROSSING STRUCTURE
84" ϕ BYPASS
WATER POLLUTION CONTROL PLANT
WPCP RIVER CROSSING SIPHON
CHLORINATION BUILDING
PLANT BYPASS CHAMBER
84" ϕ
DWENGER AVE.
WAYNE ST. INTERCEPTOR
WPCP RIVER CROSSING STRUCTURE

FIGURE 5 - WATER POLLUTION CONTROL PLANT
STORMWATER TREATMENT FACILITIES

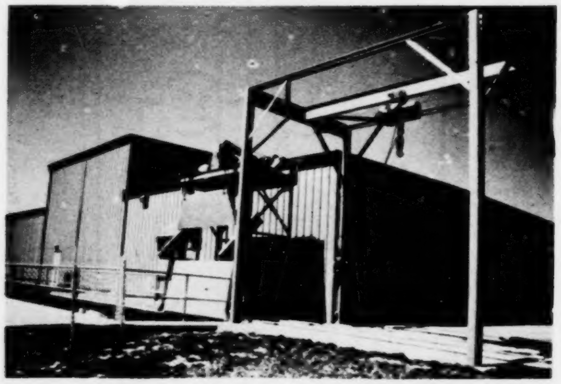


Figure 6 - Stormwater Treatment Facility
City of Fort Wayne, Indiana

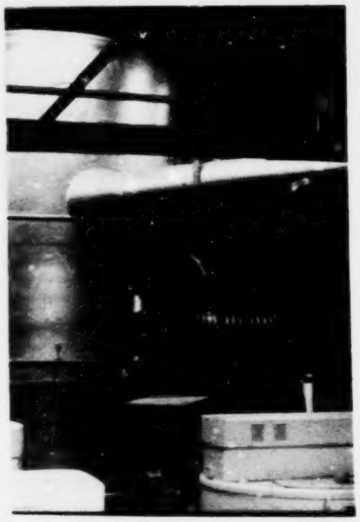
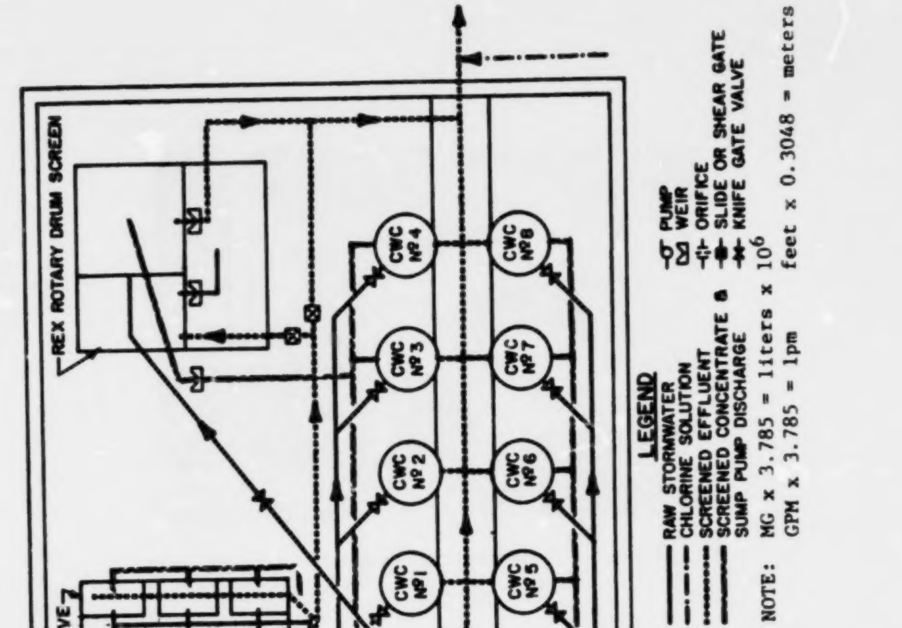


Figure 7 - Constant Head Tank Raw Stormwater
Distribution Device


REX ROTARY DRUM SCREEN
BAUER HYDRASIEVE
CWC Nº1
CWC Nº2
CWC Nº3
CWC Nº4
CWC Nº5
CWC Nº6
CWC Nº7
CWC Nº8
HEAD TANK
SUMP PUMP
18" POND RECIRCULATION LINE
STORMWATER FACILITY WETWELL (0.55 MG)
24PS PUMP 17,500 GPM
30PS PUMP 35,000 GPM
TWIN 96" RIVER CROSSING
TO LAKESIDE INTERCEPTOR SEWER
LEGEND
RAW STORMWATER
CHLORINE SOLUTION
SCREENED EFFLUENT
SCREENED CONCENTRATE &
SUMP PUMP DISCHARGE
PUMP
WEIR
ORIFICE
SLIDE OR SHEAR GATE
KNIFE GATE VALVE

NOTE: MG x 3.785 = liters x 10^6
GPM x 3.785 = lpm
feet x 0.3048 = meters

FIGURE 8 - STORMWATER TREATMENT FACILITY
FLOW ROUTING SCHEMATIC DIAGRAM

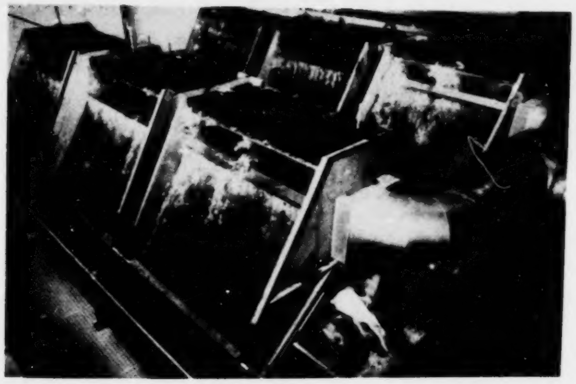


Figure 9 - Bauer Hydrasieve®
as Originally Installed

Figure 10 - Bauer Hydrasieve® as
Modified with Splash Guards
East Bauer

three of the units on one side of the east bank under full hydraulic loading. This photograph also shows the field modification splash guards to the units installed to permit greater flow distribution capability.

These screens operate on the hydraulic surface attachment principle of the Coanda effect. The solids-laden fluid enters the headbox between the two banks of modules and overflows a weir above the screens. The fluid is accelerated down the screen by gravity and the water tends to "slip through" the screen openings set at 0.060 inch (1.52 millimeters). The screened effluent drops into a common channel in each module while the solids-laden screenings fall into channels along the length of each module where it flows to a common concentrate channel.

Another screening method evaluated was the Rex Rotary Drum Screen. The Rex screen operates on the principle that screening is accomplished by forcing solids-laden wastewater through a rotating drum screen. The solids first are deposited by sieving then by straining as the filter mat builds up. The pressure gradient driving the fluid across the screen comes from the differential head developed by the raw waste influent into a rectangular headbox structure communicating into the interior of the drum. The flow moves from here to the interior of the drum through an open end and passes through the screen media on the drum circumference. After passing through the media, the screened effluent overflows an effluent weir to the effluent channel.

Figures 11 and 12 show the Rex drum screen under operational conditions. The rate of rotation and the hydraulic loading are variable for this screening method.

Screenings collected by the media are carried up and out of the flow as the drum rotates. When screenings build up on the screen media sufficiently to cause the head differential through the unit to exceed 24 inches (61 centimeters), a backwash spray header system washes the screenings from the media. The screenings drop into a collection trough inside the drum and flow by gravity to the common concentrated screenings channel.

The Rex screen installed was a 12-foot (3.65 meters) effective diameter drum screen mounted horizontally within a reinforced concrete chamber. The flow enters the screen inlet chamber in a direction parallel to the drum axis. The drum length is 12 feet (3.65 meters). There are 24 separate rectangular panels 1'-6" by 12'-0" (0.45 m by 3.66 m) mounted on the circumference of the screen for a total screen area of 432 square feet (40.1 square meters). The screen material is No. 316 stainless steel tensile bolt cloth mesh No. 100 capable of screening particles down to a minimum size of 75 micrometers.

The screen rotating speed is variable over a range of 2.8 to 11.2 rpm. The hydraulic loading rate is designed to be a maximum of 18.75 MGD (0.906 cumec), or a hydraulic loading rate of 60 gpm per square foot per minute (21.1 liters per minute per square meter per minute), based on a submergence ratio of 0.54.

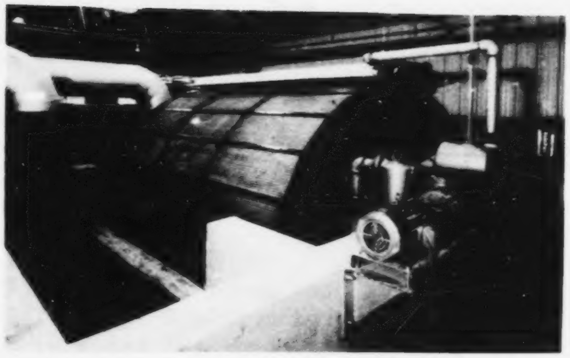


Figure 11 - Rex Rotary Drum Screen

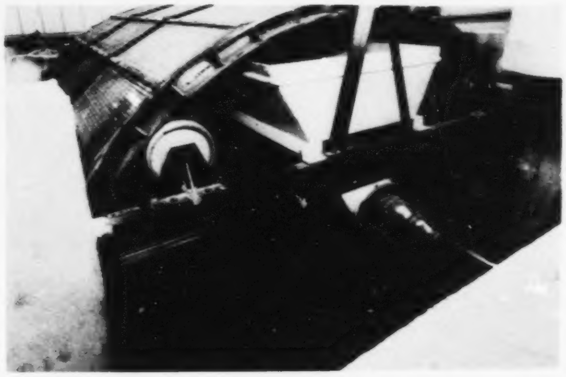


Figure 12 - Rex Rotary Drum Screen
Automatic Backwash
Collection Trough

Eight SWECO CWC® Centrifugal Wastewater Concentrators divided into two banks of four units each are installed overhanging an open effluent channel that collects the screened effluent from all the screening units plus the excess of the head tank overflow weir. These units are each 60-inch (152 cm) SWECO CWC® units. A rotating screen cage drive within a rigid circular outer steel shell forms the basis for the screening. Screening is accomplished mostly by straining although some buildup of solids removed can be noticed by excessive concentrate flows. During the operation of the screening units, the screens are continuously "washed down" by the action of the directed influent flow. This "washing" action produces the "concentrated screenings" flow stream which is collected and conveyed to the common screenings.

Figure 13 is a photograph of one of the SWECO units. The utility connections for hot and cold backwash rinse water are visible in the photograph. Figure 14 is a photograph of the manner of installation of the eight SWECO screens along the facility screened effluent channel that are capable of operation in tandem with the Bauer Hydrasieve®.

The centrifugal wastewater concentrator has a rotating collar containing 36 individual screen panels each of approximately 2 square feet (0.19 square meter) of No. 165 mesh (105 micrometer openings) screens. Each CWC unit is nominally rated at 1,500 gpm (5,700 lpm) although they have been successfully operated at up to 4,000 gpm (15,000 lpm) each. The unit is limited more by the quantity of concentrated screenings generated and the structural limitations of the screen material than other factors.

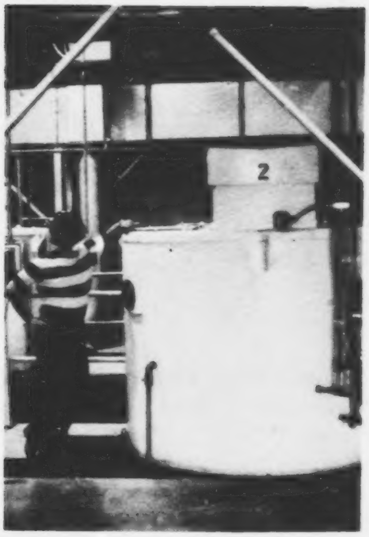


Figure 13 - SWECO CWC-60® Centrifugal
Wastewater Concentrator

Figure 14 - North Bank of SWECO CWC-60®
Screening Units

SECTION 5

EXPERIMENTAL PROGRAM

PRE-CONSTRUCTION EVALUATION PHASE

River Sampling Locations

To evaluate the effects of combined sewer overflow wastewater on the receiving stream, water quality and flow data of the Maumee River were monitored from September 1971 to December 1972. This water quality monitoring program was undertaken to evaluate the impact of untreated combined sewer overflows on the river. More than fifty separate storm events were monitored for the individual pollution loading characteristics of each event.

Three Maumee River flow measurement and sampling points were monitored in the pre-construction 1971 - 1972 sampling program.

1. Maumee River at Anthony Boulevard (one-quarter mile, 0.4 km upstream);
2. Maumee River at U.S. 30 Bypass (1-1/2 miles, 2.4 km downstream);
3. Maumee River at the New Haven Bridge (4-1/2 miles, 7.2 km downstream).

A map showing these monitoring points is shown in Figure 3.

The stormwater events in the program were assigned sequential identification numbers starting with 001 and running to 058 in the pre-construction sampling program in 1971 - 1972. After completion of the construction program, a similar numbering system started with EVENT 101 and ended with EVENT 216. All intervening events between event 059 and 100 were not considered in the study program.

A primary evaluation objective resulted from the monitoring of the impact of storm-caused overflow on the river. To evaluate the rainfall intensity factor on the overflows, rainfall data from five stations were monitored in both the pre-test and post-test situation using recording rain gages. These rain gage stations were identified with the following code locations.

Station 1 - Brown Street Lift Station - Brown Street at Maumee River on the City's west side at a distance of 3 miles (4.8 km) west of the Water Pollution Control Plant.

Station 2 - City Utilities Park - Baals Drive at St. Joseph River approximately 2-1/4 miles (3.6 km) north of the Water Pollution Control Plant.

Station 3 - Deleted.

WPCP - Water Pollution Control Plant.

Weather Bureau - U.S. Weather Service information from Baer Field, Fort Wayne. (This data is usually obtained from the U.S. Weather Service monthly weather summary.)

The Maumee River elevation at the Water Pollution Control Plant is obtained from the New Haven stream gaging station, applied back upstream. A reading of this gaging station was taken at the start of the event, and daily as part of the routine data-gathering at the plant.

The Morton Street Lift Station was selected as a major source of bypassing stormwater overflows due to its large drainage area of mostly residential neighborhoods. Elapsed hour meters were installed on the existing stormwater pumps. As the head increased in the Morton Street Lift Station wetwell, the stormwater pumps automatically started and actuated the running time clocks. The amount of stormwater bypassed from the Morton Street station was estimated from the total elapsed time and the average pumping rate derived from the wetwell depth recorder.

Another major source of the storm-caused overflow at the Water Pollution Control Plant was selected for monitoring in the pre-construction phase and later for treatment in the construction program. Installed in the 84-inch plant bypass structure was a water level recorder in which the depth of the stormwater in the chamber triggered a depth recording chart in the Water Pollution Control Plant control room. The depth of water in this structure provided an estimate of the total raw stormwater being diverted to the river in the pre-construction phase. Later, this same monitoring point was used to indicate the start of a stormwater treatment event after the screening facility became operational.

Pre-Construction Sampling Program

The objectives of this program called for determining the time-dependent characteristics of the stormwater overflow. This was accomplished by taking grab samples and flow measurements at predetermined time intervals. The sampling program was intended to start with the initial storm-caused overflow. This condition was termed "first flush", and the constraint upon the sampling was to take a grab sample within the first 15 minutes after the overflow began (determined from the 84-inch bypass depth chart at the Water Pollution Control Plant).

Sampling was usually continued to the point where the discharge to the river ceased altogether and the pollutional loading of the river dropped to normal levels. As a practical matter, on several events, it might have taken days for the river discharge to return to the normal dry weather flow condition. The decision to terminate an event sampling program was made when at least two sets of river samples had been obtained with one of them taken within the interval of overflow bypassing. Sampling on occasion continued beyond the end of the bypass condition in response to some evidence that the peak pollution loading was still building up. Normally, however, the end of the overflow in the pre-construction phase signaled the end of the event sampling.

River samples were taken at the three points every hour or so after the start of the event. The method of sampling was to obtain a grab sample of the main stream flow. The samples were field-analyzed for pH, temperature, fecal coliform, and dissolved oxygen. Other analyses were run in the plant laboratory on turbidity, chemical oxygen demand, biochemical oxygen demand, suspended solids, volatile solids, ammonia nitrogen, and total phosphorus. Staff gage observations were made at the same time as the physical and chemical samples taken at the Anthony Boulevard Bridge where the Weather Bureau maintains a stream gaging station.

The duration of river sampling in the pre-construction program normally lasted as long as the duration of the observed overflows at the Water Pollution Control Plant, with one or more sets of samples taken thereafter.

Routine plant composite sampling on the basis of daily analyses for regulatory agency reports also continued throughout the pre-construction phase. The data for the terminal pond effluent represent the best estimate of plant pollutional loadings to the river over this period of time.

Table 1 is provided to summarize the sampling locations used and the analyses required for the pre-construction evaluation phase.

POST-CONSTRUCTION EVALUATION PHASE

Screening Equipment Locations and Flow Measurements

In the evaluation of the screening equipment as stormwater treatment devices in the post-construction phase, the decision on when to start the stormwater pumps at the test facility resulted from the indication on the 84-inch bypass level indicator. When the water level in the 84-inch bypass structure was noted as reaching the point where a depth of water showed on the bubbler depth recorder in the control room, the Water Pollution Control Plant operator notified the project director. At the point of incipient overflow of the 84-inch bypass, the water level indicator at the Stormwater Facility wetwell would read approximately 15 feet.

Specially-trained operators were alerted and directed to their work stations for the duration of the stormwater treatment event. One such person, the river sampler, was responsible for driving the road circuit taking river samples at the Anthony Boulevard Bridge, the U.S. 30 Bypass Bridge and the New Haven Bridge. The Stormwater Facility operator was stationed at the

Table 1. PRE-CONSTRUCTION EVALUATION PHASE SAMPLING PLAN
August, 1971 to November, 1972

Sampling Location	pH	Temp. (°C)	Settleable Solids (ml/1)	Fecal Coli. (100 ml)	Turb. (JTU)	COD (mg/1)	SS (mg/1)	VSS (mg/1)	NH_4^+-N (mg/1)	D.O. (mg/1)	BOD_5 (mg/1)	Total P (mg/1)	Stream Gage (Ft)	Flow (GPM)
Primary Effluent	X	X		X		X	X			X	X	X		X
Secondary Effluent	X	X		X		X	X			X	X	X		X
Terminal Pond Effluent	X	X		X		X	X	X		X	X	X		X
84-inch Bypass Effluent	X	X	X	X		X	X	X			X	X		X
Morton St. L.S. Effluent	X	X		X		X	X	X			X	X		X
Maumee River Anthony Blvd. 1/4 mile upstream	X	X		X	X	X	X			X	X	X	X	X
Maumee River U.S. 30 Bypass 1-1/2 mile downstream	X	X		X	X	X	X			X	X	X	X	X
Maumee River New Haven 4-1/2 mile downstream	X	X		X	X	X	X			X	X	X	X	X

Note: All sampling is based on grab samples except the Primary Effluent, Secondary Effluent, and Terminal Pond Effluent which are based on 24-hour composite samples.

stormwater pump control room to operate the stormwater pumps and screening devices, communicate with the Water Pollution Control Plant, and direct the sample-gathering at the facility. Normally, a third person was assigned to the facility to assist the operator in taking samples at the various locations in the sampling plan.

The project director usually supervised the Water Pollution Control Plant operators in flow-routing and raw sewage pumping at the plant. This activity was usually necessary because the plant personnel had been using the volume of the plant influent sewers for raw wastewater storage and flow equalization for years prior to this work. Careful flow-routing in the plant and sequential raw sewage pumping were necessary to avoid process problems with the plant when stormwater-induced surges in hydraulic flow occurred.

The decision to begin an event rested with the project director exclusively. Sometimes, the storage volume of the plant interceptors filled the river crossing conduits enough to show on the 84-inch bypass bubbler chart even when there was no rainfall preceding the diversion. Until the wetwell level at the Stormwater Facility reached 26 feet on the depth indicator there, it was impossible to bypass raw sewage to the river without some kind of treatment.

The sequence of events preceding a pumping cycle typically involved manning the facility as described above. The operator was instructed to start one of the stormwater pumps when the wetwell level reached 20 feet on the bubbler level indicator. The decision of whether to use the 24 PS (17,500 gpm, 1.096 cumec) or 30 PS (35,000 gpm, 2.101 cumec) pumps usually was made on the basis of equalizing the time of operation for each pump.

Once the pumping of the wetwell contents began, the project director decided whether to sample the flow or to merely "pump down" to clear the wetwell and river crossing conduits of the raw wastewater. In the post-construction sampling period commencing in January, 1975 and ending in February, 1976, there were 116 bona fide events triggered by the depth indicator in the 84-inch bypass. There were samples taken in 53 events, not all of which were stormwater-induced. Some of these events were initiated by diverting plant raw sewage to the Stormwater Facility by shutting down one or more raw sewage pumps and allowing the raw wastewater to fill up the river crossing to the stormwater station. Some other events were the result of recirculating Terminal Pond No. 1 contents. Most events were the result of stormwater-induced flows in the combined sewer collector areas, however.

The raw wastewater pumped by the stormwater pumps discharged into a 14'-0" (4.27 meter) diameter head tank (32,500 gallons, 123,000 liters capacity) before being distributed to the treatment units.

The head tank acted as a surge tank for the stormwater pumps, as a constant head flow control device for the screening equipment, and as a completely mixed sampling vessel for obtaining the raw wastewater sample aliquots needed to perform the evaluations.

Figure 8 shows a schematic diagram of the flow routing to the different screening devices as well as the overflow points necessary to protect the screening room equipment from excessive flow surges and capacity. The head tank has a 6'-0" (1.83 meter) diameter overflow weir discharging excess raw wastes pumped to the head tank directly into the screened effluent channel. By means of a water surface level indicator on the side of the head tank, sampling personnel were able to determine the depth of wastewater in the head tank at any time. This observation provided the measurements necessary to determine the quantity of raw wastewater pumped to the head tank and bypassed over the 72-inch circular overflow weir.

The facility was designed to evaluate the relative treatment efficiency of three different types of screening devices on stormwater-induced combined sewer overflows. The flow split to these screening devices was determined by dividing the total pumping capacity into four segments. The intent of this arrangement was to apportion the available hydraulic flow equally to each of the screening devices installed. The channels for this purpose were designed in such a manner as to provide ample hydraulic head to run the Bauer Hydrasieve® fixed screens in tandem with the Rex Rotary Drum Screen and the SWECO Centrifugal Wastewater Concentrators. In other words, a dual-use objective of using the Bauer screens as a pretreatment device for both the Rex and SWECO screens was built into the flow diversion conduits.

Because determination of treatment efficiency was a prime objective from the outset, accurate flow measurements for each screening device were essential. The project sampling and flow measurement protocol established flow measurement weirs, staff gages, and orifices that were to be calibrated with primary flow measurement devices, and used thereafter for making field observations that were treated as secondary flow measurements. A calibration deviation of ±10 percent from the actual measured flow was considered adequate to meet the treatment evaluation program objectives within the evaluation phase budget for the project.

Referring to Figure 8 again, there are five discharge points out of the head tank. One is the 72-inch overflow weir discussed above. On the south side of the tank, a 24-inch diameter line feeds a bank of four SWECO CWC® Centrifugal Wastewater Concentrators. A flow measurement of the head tank discharge feeding this bank was made using a 16-inch diameter sharp-edged orifice installed at the flange in the line. Arrangements were made to equally divide the flows to the individual units by balancing the influent flows using a pressure drop measurement across the 12-inch inlet elbow to each unit. The influent gate valve slide position was adjusted in the field to equalize this pressure drop. Thereafter, measurements of flow to this bank of units was made by observing a mercury-filled U-tube manometer attached to centerline pressure taps upstream and downstream (at the vena contracta point) from the orifice. Computations of the hydraulic discharge corresponding to these measurements were done using the adjusted orifice equation, $C = 0.60$.

Another bank of SWECO CWC® units was installed on the other side of the screened effluent channel. This bank had the capability of being operated as primary treatment devices or secondary treatment behind the Bauer unit as a pretreatment device. Except for this feature, the measurements of flow for this installation were identical to the south bank.

The Bauer Hydrasieve® units were arranged in two banks of 72-inch wide screens, with six such screens on each side, east and west. There was no way to vary the flow to either side except to throttle the influent gate valve and thereby restrict the flow to the entire unit. However, it was observed in the field that the velocity of the discharge from each bank was equal at three different flow rates. Therefore, a rectangular weir was installed in the discharge section of the west bank unit with a staff gage located approximately four feet (1.2 meters) behind the weir. The operator was instructed to take a staff gage reading at intervals set in the sampling protocol on the west bank Bauer unit at the same time he was taking a reading on the head tank water level. The assumption that there was no difference between the (measured) west bank units and the (assumed equal) east bank units was validated by field measurements of discharges in the effluent channels of both tanks.

The Rex flow measurements used the built-in 132-inch (3.35 meters) rectangular weir at the effluent side of the screening equipment. A staff gage was placed behind the weir and a float sensor head recorder in a stilling well was also installed.

To protect the drum from damage resulting from hydraulic overloading of the unit, a 36-inch (0.9 meter) rectangular weir was installed in the headbox for the unit to allow raw wastes to fill up the interior of the drum to the overflow elevation. A staff gage and head recorder in a stilling well were installed also. Flow control to this unit was possible by throttling the influent gate valve coming from the head tank.

Measurement of the screenings discharge of the treatment units was also necessary. A series of rectangular channels in the floor of the screening equipment room of the facility were utilized by placing standard suppressed rectangular weirs at the ends of the SWECO collection channels with staff gages. Operators were instructed to measure down from a convenient reference point (the floor channel grating) to the water surface. The investigators intended these observations to be used as screenings flow measurements by converting these observations to staff gage datum. Unfortunately, most of the stormwater-induced events resulted in submerged weirs due to hydraulic back head from the surcharged effluent sewer line. Therefore, few flow measurements for the SWECO screening concentrate were considered valid enough to be used in evaluation. Rather than using measurements that were likely to grossly over-estimate these flows, the flow measurement protocol was altered to use assumed screenings flows in place of these field observations.

The flow of screenings from the Rex drum screen was conveniently measured by a weir box with a 60-degree V-notch weir installed. Operating personnel were instructed how to measure the head over such a device and the results recorded on a field data sheet. These head measurements were converted to discharge measurements by using the standard 60-degree V-notch weir tables.

A 108-inch (2.74-meter) diameter circular weir was designed to operate as a fully developed circular weir for flow measurement checks using a staff gage and/or head recorder at the scum baffle a few feet upstream from the stage monitoring point. This stage level measurement technique was field-calibrated by a current meter calibration procedure in April, 1975 using a cable-suspended Price-type current meter. This calibration was necessary to provide an independent flow measurement as a base line data point to permit accurate estimates of flow measurement error.

Two other staff gage measurement points were set in the screened effluent channel. One point was set at the discharge end of the screening room open channel exiting the building. Another point was located 550 feet (167.6 meters) downstream from the facility building. These two points were field-calibrated at the same time as the circular weir at the end of the channel.

Raw sewage pumping rates were determined from the pump head discharge curves. The pump operating point was calculated from observed levels of the Stormwater Facility wetwell indicator. The amounts calculated from this source were compared with the sum of the amounts measured, with due consideration to the problem of relative error. The chlorine contact channel calculated discharge from the observed head gage readings was used as a last-resort check of these quantities to help sort out discrepancies in the flow measurements. In this way, an estimate of maximum probable error was made for those events where the data show anomolous trends of behavior.

Other flow measurements used in this evaluation program came from routine flow observations and records of the plant. Specifically, the terminal pond influent coming across from the Water Pollution Control Plant was taken from the raw sewage flow meter at the plant. The terminal pond effluent flow was derived from the measured raw sewage flow at the WPCP and the measured stormwater treatment flow at the chlorine contact channel, using the 8'-0" (2.44 meter) rectangular sluice gate opening as a conventional broad-crested suppressed rectangular weir. Knowing the pond depth, and therefore, the head over the "weir", a pond discharge was calculated.

Post-Construction Sampling Program

The parameters intended to be measured in the post-construction program were selected from the experience of the pre-construction evaluation phase sampling plan (See Table 1, preceding). Table 2 represents a summary of the post-construction evaluation schedule of sampling and analysis. Note that there were additional parameters added in the post-construction phase evaluation; namely, chlorine demand, residual chlorine, odor, total organic nitrogen, as well as ten additional sampling locations.

Evaluation of the screens was based on the difference between the raw wastewater loadings and the treated wastewater loadings. Multiple samples were taken at predetermined time intervals because the objectives of this program called for following a quantity of stormwater overflow through the treatment processes and eventually by discharge to the stream from the terminal pond effluent. There, grab samples were mostly used in this phase.

Table 2. POST-CONSTRUCTION EVALUATION PHASE SAMPLING PLAN
January, 1975 to February, 1976

Sampling Location	pH	Temp. (°C)	Settleable Solids (ml/l)	Fecal Coliform (N/100ml)	Turbidity (JTU)	Chemical Oxygen Demand (mg/l)	Suspended Solids (mg/l)	Volatile Suspended Solids (mg/l)	NH_4^+-N (mg/l)	Chlorine Demand (mg/l)	Chlorine Residual (mg/l)	Dissolved Oxygen (mg/l)	Odor	BOD_5 (mg/l)	Total P (mg/l)	Total Organic Nitrogen (mg/l)	Staff Gage (ft)	Flow (GPM)
Primary Effluent	X		X	X		X	X	X	X	X	X	X		X	X			X
Secondary Effluent	X		X	X	X	X	X	X	X	X	X	X		X	X			X
Plant Raw Sewage	X	X	X			X	X	X	X					X	X	X	X	X
Head Tank	X		X	X		X	X	X	X				X	X	X			X
Bauer Screened Effluent	X			X		X	X	X	X	X				X	X		X	X
Bauer Screenings Concentrate	X			X		X	X	X	X					X	X		X	X
Rex Screened Effluent	X			X		X	X	X	X	X	X			X	X		X	X
Rex Screenings Concentrate	X			X		X	X	X	X					X	X		X	X
SWECO CWC Screened Effluent	X			X		X	X	X	X	X	X			X	X		X	X
SWECO CWC Screenings Concentrate	X			X		X	X	X	X					X	X		X	X
Combined Screenings Facility Concentrate	X			X		X	X	X	X				X	X	X		X	X

Table 2. POST-CONSTRUCTION EVALUATION PHASE SAMPLING PLAN
January, 1975 to February, 1976 (Continued)

Sampling Location	pH	Temp. (°C)	Settleable Solids (ml/1)	Fecal Coliform (N/100ml)	Turbidity (JTU)	Chemical Oxygen Demand (mg/1)	Suspended Solids (mg/1)	Volatile Suspended Solids (mg/1)	NH_4^+-N (mg/1)	Chlorine Demand (mg/1)	Chlorine Residual (mg/1)	Dissolved Oxygen (mg/1)	Odor	BOD_5 (mg/1)	Total P (mg/1)	Total Organic Nitrogen (mg/1)	Staff Gage (ft)	Flow (GPM)
Chlorinated Screened Effluent Channel	X		X	X		X	X	X	X					X	X	X	X	X
Terminal Pond Effluent	X	X	X	X		X	X	X	X		X	X		X	X			X
Maumee R. Anthony Blvd. 1/4 mile Upstream	X	X			X		X	X				X		X	X		X	X
Maumee R. U.S. 30 Bypass 1-1/2 mile Downstream	X	X			X		X	X				X		X	X		X	X
Maumee R. New Haven 4-1/2 mile Downstream	X	X			X		X	X				X		X	X		X	X

Some of the points where aliquots were to be taken in the screening facility represented more than one physical point of sample collection. For example, there were eight SWECO Centrifugal Wastewater Concentrators (CWC) located in the screening equipment room of the building. Each CWC contributed its own effluent and screenings to a common channel designed to carry all the screenings and effluent away as they were discharged. Other treatment units also discharged into these channels, and the individual sampling of these incremental discharges would have presented an impossible analysis burden on the plant laboratory personnel. Therefore, for specified sampling points, the operators were instructed to take equal portion aliquots from each operating unit and composite these as the "grab" sample of the location required in the sampling plan.

The single raw sewage sampling point was located at a sampling port at the head tank. A sample was drawn from this sample port into a clean sample container. Field measurements of pH, temperature, D.O., and staff gage water levels were made on the spot and recorded. Grab samples were taken according to the Schedule of Laboratory Analyses worked out in the pre-construction effort. Analyses run on these grab samples were fecal coliform, turbidity, COD, suspended solids and volatile suspended solids. A single composited sample made up two or more equal-sized aliquots taken at the same time as the grab samples were analyzed for BOD_5, ammonia nitrogen, total phosphorus, and pH.

Each of the screening equipment effluent samples and flow measurements was analyzed separately, within the constraints of multiple sampling points outlined above. The following equipment was sampled for a single grab sample per device for each sampling interval: Bauer East, Bauer West, Rex SWECO South, and SWECO North. Parameters measured in the field were pH, temperature, D.O. and staff gage and/or manometer readings, as appropriate. Grab samples were also taken one per interval for the same parameters required of the head tank. Similarly, for the composited samples, a single sample per interval was obtained.

A single screening equipment concentrate sample was taken at each of the Bauer and Rex units. A concentrate channel for the SWECO's was sampled by composited aliquots in an attempt to obtain a representative sample for the time interval involved. Individual samples and flow measurements were also analyzed for the same scope of analyses as the head tank.

The combined screenings were also measured to determine the impact of the screenings collected on the Water Pollution Control Plant. These measurements were conducted along the same lines as the head tank above.

Another sampling point was located at the overflow weir of the chlorine contact channel. This point, designated as "Chlorinated Screened Effluent Channel", was monitored for all of the analyses covered under the head tank plus the addition of residual chlorine.

Routine plant performance monitoring was already being done by collection of 24-hour flow-composited information of the type being sought at the Stormwater Treatment Facility. These data were intended to be used to identify the impact of the screening facility on the plant and to determine the relative effect on the river by rerouting some previously bypassed stormwater overflow to the plant treatment lagoons. The plant data collected were flow-composited manually from aliquots taken at several sampling locations including the raw sewage, primary effluent (in excess of the secondary treatment capacity of 32 MGD), secondary effluent, chlorine contact tank effluent (consisting of secondary effluent plus the diverted primary treated secondary bypass flow), and the terminal pond effluent. Daily analyses were run on these samples for all of the analyses required of the head tank plus chlorine demand and residual chlorine as appropriate.

One of the original objectives of the program was to determine the individual screening units' effectiveness in dealing with the anticipated heavily-polluted "first flush" overflows that were observed in the pre-construction evaluation program. Toward this end, each of the screening devices' individual sampling protocol called for taking the first four samples not more than fifteen minutes apart for the first hour after the start of the event, not more than thirty minutes apart for the next five hours, and not more than sixty minutes apart thereafter to the end of the events. However, in the post-construction evaluation period commencing in January, 1975 and running until February 6, 1976, there were not even two or three events that even remotely resembled the characteristics of a "first flush" phenomenon. The decision was made in June, 1975 to dispense with the sampling and analysis burden of attempting to catch the so-called "first flush", and the post-construction sampling and analysis protocol was modified accordingly.

Screening Equipment Efficiency Evaluations

Each of the screening equipment suppliers was required, as a part of the bids, to submit a statement regarding the equipments' anticipated performance screening combined sewer overflow wastewater. From these statements, a method of evaluation consistent with the operation of each type of equipment was devised.

Each of the screening equipment suppliers claimed a degree of removal efficiency on the pollutant parameters, biochemical oxygen demand and suspended solids. In order to evaluate removal efficiencies on a consistent, objective basis it was necessary to use these pollutant loadings (and others) rather than concentrations as the measure of before treatment - after treatment. This resulted in the computation of influent pollutant loadings for each screening method separately. For this computation, the raw stormwater pollutant concentration was used along with the time interval measured instantaneous flow rate to the individual screening equipment. A measured effluent flow rate (where possible) times the sampling results pollutant concentrations provided the effluent loading parameter. A computer program was written to perform the incremental computations necessary to determine these loadings and the incremental results were tabulated and summed in the computer output table. The computer also tabulated the pollutant concentrations by time of sampling and location, and provided a table of sampling and analyses results.

Efficiency of removal was calculated from the computer output mass loadings by performing the following computation for each equipment location or unit process considered:

$$\begin{array}{c}\text{Efficiency}\\ \text{of Removal}\\ \text{(percent)}\end{array} = \frac{\left(\begin{array}{c}\text{Influent}\\ \text{Pollutant}\\ \text{Loading}\end{array}\right) - \left(\begin{array}{c}\text{Effluent}\\ \text{Pollutant}\\ \text{Loading}\end{array}\right)}{\left(\begin{array}{c}\text{Influent}\\ \text{Pollutant}\\ \text{Loading}\end{array}\right)} \times 100$$

The pollutants for which this computation was performed include BOD_5, COD, total organic nitrogen, ammonia nitrogen, total phosphorus, suspended solids, volatile suspended solids, chlorine demand and residual chlorine. The sampling locations where the mass loadings were computed included the Maumee River at Anthony Boulevard, U.S. 30 Bypass, and New Haven, the Stormwater Facility head tank, Bauer screened effluent east and west, Rex screened effluent, Rex screening concentrate, combined screening facility concentrate, and the chlorinated screened effluent channel, the Water Pollution Control Plant primary effluent (bypassing secondary treatment), secondary effluent, and Waste Stabilization Pond No. 1 effluent.

Stormwater Treatment Facility Impact on Water Pollution Control Plant

This evaluation result was necessary because the existing 60 MGD Water Pollution Control Plant (WPCP) was expected to treat the combined sewer overflow Stormwater Facility concentrate at the conventional activated sludge plant on the south side of the Maumee River. In addition, the screened effluent was added to the contents of Terminal Pond No. 1 which had been already receiving the wastewater from the WPCP since October, 1971.

The evaluation of the impact of the Stormwater Facility concentrate on the WPCP was anticipated to be done quantitatively by measuring the flow rate and relevant pollutant concentrations at the facility itself, and comparing with the WPCP raw sewage results for the same time frame. However, with the problems measuring the concentrate flow rate cited earlier and the observed low pollutant concentrations at the Stormwater Facility concentrate channels, the decision to proceed with this evaluation on a semi-quantitative basis was made. The loadings indicated for this part of the evaluation are based on assumed flow rates and should be considered as semi-quantitative.

The effect of chlorinated combined sewer overflow screening effluent was an integral part of the overall treatment capability evaluations anticipated for this project. Plant operators had been taking daily composited samples of the terminal pond effluent and running the pollutant analyses for BOD_5, COD, suspended solids, volatile suspended solids, total Kjehldahl nitrogen, total phosphorus, and pH. Grab samples for the pond effluent D.O., fecal coliform, and residual chlorine were also taken since the pond start-up.

Trend analysis of the pond data available from these routine plant operating and monitoring functions was expected to provide some of the results needed to evaluate the effect of the screened, chlorinated stormwater overflows on the pond. In addition to these data, observations by the plant operators of algae growth in the pond, qualitative operational variations such as floating scum and grease, and other special tests on the pond effluent were expected to provide added performance data.

Dry-weather screening of the pond to evaluate these screens on algae-laden pond contents was also a requirement of the program. An 18-inch pond recirculation line was constructed as a part of the facility construction grant to permit this dual-use mode.

Determination of Shock and Total Pollutional Loads on the Receiving Stream

The effect of the raw, untreated combined sewer overflows on the receiving stream was the major objective of the pre-construction evaluation phase. In the construction phase, the installation of conduits and structures to divert the 84-inch bypass overflow to the Stormwater Facility was intended to essentially eliminate this source of overflows directly to the Maumee River. The post-construction evaluation was intended to quantitatively measure the same stream water quality parameters after completion of construction to determine if any improvement could be measured. A trend analysis approach should also indicate any measurable improvements in the stream water quality, particularly in terms of maximum and/or minimum pollutant parameters.

In substantiation of the anticipated improvements in stream water quality, the effects of the terminal pond effluent itself was required to assess the changes measured. If the Stormwater Facility effluent could be shown to have minimal effect on the pond, and the pond could be shown to be a minor source of stream pollutants before and after the construction program, a measurable relative improvement in stream water quality downstream from the location of the former 84-inch bypass for combined sewer overflows would indicate a significant improvement in stream water quality that could be attributed in whole or in part to the Stormwater Facility.

Screening Methods Cost Analysis

A primary objective of this program was to determine the costs of screening operations treating the previously bypassed raw combined sewer overflows. In order to do this, all of the construction and operations costs of the facility were monitored to provide costs data for the evaluation of the Stormwater Facility. However, to distribute these costs to the different screening methods used in the evaluation required some estimations of power usage for the stormwater pumps, the facility lighting and heating equipment, and the different screening methods.

Pumping horsepower requirements for the flows delivered for each event were to be allocated to the different screening equipment by the use of a flow proportionally constant (determined from the ratio of the discharge treated to the total discharge pumped). To this power requirement to simply deliver the raw stormwater overflow to the individual screening devices was added the computed horsepower directly required to operate the screening device. All such horsepowers were used to calculate the kilowatt-hour power requirement which was summed and used to compute the operating costs of this screening method. Added to this cost were the costs of potable water usage, chemical consumption, etc.

Labor costs were considered to be more of a reflection of the effort expended to gather samples and make observations for completing the R/D evaluation of this program rather than an accurate rendering of actual operational manpower requirements. Therefore, labor costs are not included in costs derived for this program.

Once all the costs for an event were estimated, a cost per million gallons treated was calculated. This result was to be tabulated as an evaluation parameter just as treatment efficiency was to be used. This resulting dollars per million gallons figure was also summed and averaged, and the overall costs estimated to be compared with the actual costs incurred. Any discrepancies would have to be analyzed and explained under this program.

SECTION 6

RESULTS OF THE EXPERIMENTAL PROGRAM

STREAM WATER QUALITY IMPROVEMENTS

Introduction

The Maumee River is a tributary stream to Lake Erie. The City of Fort Wayne is the largest municipality located in the drainage basin of the Maumee River. In the "Lake Erie Report"[1], prepared by the U.S. Federal Water Quality Administration, the City of Fort Wayne was cited as a major source of wastewater to the Maumee River. A significant part of the problem cited in this report was reportedly the extensive municipal combined sewer system in the City with multiple high flow diversion bypass points located throughout the City.

A major objective of this research, development and demonstration grant program was to determine the benefits to the receiving water of diversion and treatment of previously-bypassed combined sewer overflows. The manner of assessing these benefits has been outlined in the discussion of the experimental program, preceding. The results of this assessment are discussed for the parameters undertaken in turn.

Dissolved Oxygen

The Maumee River downstream of the City of Fort Wayne was not particularly threatened by low levels of dissolved oxygen (D.O.) in the years preceding this construction program. Nevertheless, there have been times when the stream D.O. was low enough to cause fish kills in the river. These occasions usually were the result of sustained low flow periods where a significant part of the stream flow originated from wastewater discharges upstream. These discharges were usually not associated with stormwater-induced overflows in these periods. However, it seemed logical to assume that some oxygen-demanding matter settling out of storm-caused overflows would adversely affect stream D.O. at low flow conditions. If this were the case, the stream D.O. after construction should show some improvement that might be attributed to the removal of storm-caused overflows of putrescent settleable solids. This result should show its effect most noticeably in the low flow months of the year. Once the deposited matter is settled out, it would remain in the stream sediment long after the wet weather had ended. This settled matter should exert an oxygen demand year-round, according to this thinking.

One of the objectives of the evaluation program was to attempt to show an improvement in the stream water quality resulting from the removal of a portion of the total combined sewer stormwater overflows for the City of Fort Wayne. Only approximately 5 percent of the total quantity of stormwater-induced combined sewer overflows were diverted and treated in this construction program. Nevertheless, it was hypothesized that there still should be a significant improvement in the stream water quality because this admittedly small proportion of the total overall overflow discharge was thought to be the "dirtiest". This assumption resulted from the observation that the initial rush of storm-caused combined sewer flows received at the Water Pollution Control Plant (WPCP) were frequently heavily laden with sediments, putrescibles, devoid of any D.O., and highly polluted with pathogens and other contaminants.

This so-called "first flush" tended to load the plant with this hard-to-treat polluted wastewater. These wastewaters adversely affected treatment plant operations and operators sometimes allowed these heavily contaminated wastewaters to overflow to the river via the WPCP bypass rather than attempt to treat these wastes. It was reasoned that to divert these raw wastes to a treatment facility designed especially for stormwater treatment plus detention would greatly alleviate this loading on the stream.

The pre-construction phase sampling program provided for sampling of the Maumee River upstream from the plant to establish a stream flow "baseline" to evaluate the data taken downstream. This sampling point was located at the Anthony Boulevard Bridge approximately one-quarter mile upstream from the plant's eventual discharge at the Terminal Pond No. 1.

To assess the immediate effect of the pollutional loading of the terminal pond effluent on the river, another sampling point was established at the U.S. 30 Bypass Bridge at about one-and-one-half miles downstream from the pond effluent. By the time the pond flow had mixed with the river flow over this distance, the effect of the plant discharge on the river could be measured.

A third sampling point was located approximately four-and-one-half miles downstream. The New Haven Bridge over the Maumee River was the sampling point for the flow that would travel from 0.5 to 5 hours from the pond discharge to the New Haven Bridge depending on stream flow.

The pre-construction evaluation plan was to sample at regular intervals during the time when the plant 84-inch bypass was registering flow going to the river. The plant interceptor depth was noted as WPCP Chart No. 8. At least 3.5 feet (1.07 meters) of depth in the interceptor was required to reach the 84-inch bypass sewer invert. The 84-inch bypass structure also had a bubbler tube depth indicator set to indicate when there was water in this sewer. Normally, the river water surface elevation was also recorded because a high water backwater to the plant bypass structure would sometimes fill the sewer with river water.

Pre-construction sampling results throughout the period starting in August, 1971 and ending in November, 1972 demonstrated a progressive decrease in D.O. from Anthony Boulevard to the New Haven Bridge. Dissolved oxygen data averaged for these three sampling points as shown in Table 3.

Table 3. PRE-CONSTRUCTION EVALUATION PHASE STREAM DISSOLVED OXYGEN
August, 1971 to November, 1972

	Anthony Boulevard (1/4 mile upstream)	U.S. 30 Bypass (1-1/2 miles downstream)	New Haven (4-1/2 miles downstream)
Arithmetic Mean Dissolved Oxygen (mg/l)	8.60	7.90	7.40
Standard Deviation (mg/l)	2.47	2.76	3.12
Number of Observations	639	629	653
Range of Values Observed (mg/l)	4.0 to 13.6	4.2 to 13.1	2.1 to 13.1

The ranges of values of the stream D.O. also provide some insight into the amount of oxygen-demanding materials in the period of sampling. Since stream D.O. is dependent on temperature, and the most critical period for indigenous aquatic life is during the warm weather months when the saturation D.O. is lowest, a statistical distribution of warm weather measurements shows an interesting trend. The lowest value noted in the time prior to the construction of the Stormwater Facility was at New Haven where the warm weather, low flow D.O. was 0.5 mg/l on several occasions. See Figure 15 for this trend.

Post-construction sampling of the stream water quality at the same points in the period commencing January 1, 1975, and ending February 6, 1976, resulted in consistently higher dissolved oxygen measurements for this period. Still apparent from these data is a decreasing trend of values from Anthony Boulevard to U.S. 30 Bypass to New Haven. For comparison with the pre-construction evaluation data, Table 4 displays the stream dissolved oxygen data for these same three sampling points:

Table 4. POST-CONSTRUCTION EVALUATION PHASE STREAM DISSOLVED OXYGEN
January, 1975 to February, 1976

	Anthony Boulevard (1/4 mile upstream)	U.S. 30 Bypass (1-1/2 miles downstream)	New Haven (4-1/2 miles downstream)
Arithmetic Mean Dissolved Oxygen (mg/l)	9.86	9.18	8.93
Standard Deviation (mg/l)	2.53	2.56	2.82
Number of Observations	240	234	239
Range of Values Observed (mg/l)	6.1 to 16.0	4.4 to 14.0	3.8 to 13.6

It is apparent from the examination of Table 3 for the Pre-Construction Evaluation next to Table 4 for the Post-Construction Evaluation that there has been a significant increase in the average values for stream dissolved oxygen from 1971-72 to the 1975-76 evaluation period. Figures 15 through 17, inclusive, show the progressive stream average monthly dissolved oxygen for the five-year period starting in January, 1971 and ending in December, 1975 for each of the stream sampling points in this program.

Indiana Water Quality Criteria applicable to the Maumee River and pertaining to dissolved oxygen state, in part:

> "...the following criteria are for the evaluation of conditions for the maintenance of a well-balanced, warm water fish population...applicable at any point in the waters outside the mixing zone (of the effluent and the stream):"
>
> "(1) (Dissolved Oxygen) Concentrations of dissolved oxygen shall average at least 5.0 mg/l per calendar day and shall not be less than 4.0 mg/l at any time."

From a statistical analysis of the pre-construction and post-construction, the percent of the days exceeding the minimum water quality standards was shown to be:

Table 5. NUMBER OF DAYS STREAM DISSOLVED OXYGEN LESS THAN 4.0 mg/l

	Pre-Construction Evaluation	Post-Construction Evaluation
Maumee River at Anthony Boulevard (1/4 mile upstream)	6.9	0.04
Maumee River at U.S. 30 Bypass (1-1/2 miles downstream)	49.6	0.37
Maumee River at New Haven (4-1/2 miles downstream)	23.0	11.0

Close examination of Figures 15, 16, and 17 indicates the long-term trend of dissolved oxygen appears to be improving from the pre-construction phase (1971-1972) to the post-construction phase (1975-1976). The range of values for D.O. have consistently decreased over this time period. The same relative trend is shown at all three locations. Furthermore, the corresponding long-term trend for the terminal pond D.O. shows a contrary trend to these, one in which the terminal pond originally improved in effluent D.O. in late 1971 and throughout 1972 until a more variable average D.O. was apparent in 1975-1976. All of these factors tend to indicate that there has been an improvement in stream water quality over the period that may be attributed in part or in whole to the diversion and treatment of combined sewer overflows.

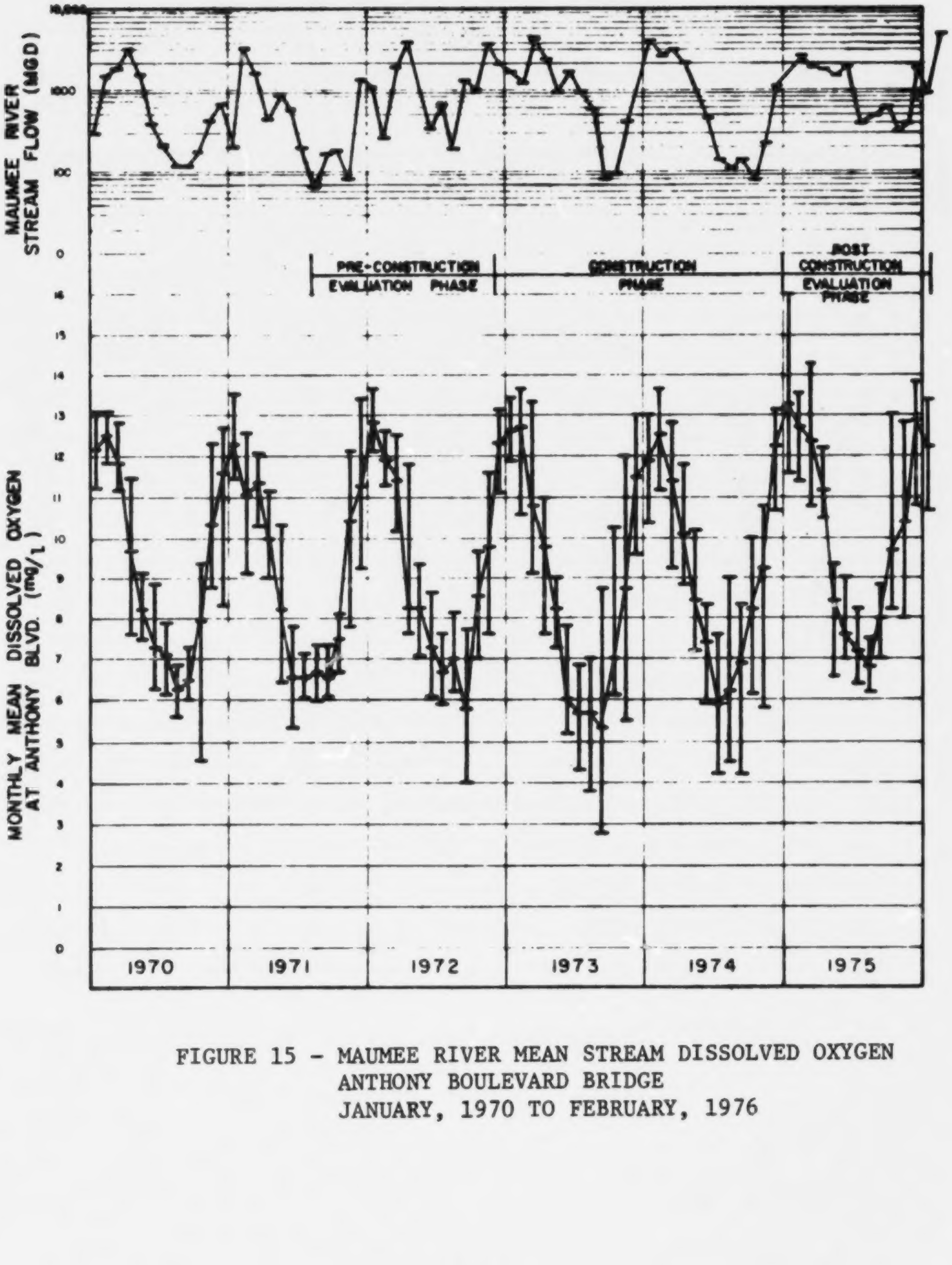


FIGURE 15 - MAUMEE RIVER MEAN STREAM DISSOLVED OXYGEN
ANTHONY BOULEVARD BRIDGE
JANUARY, 1970 TO FEBRUARY, 1976

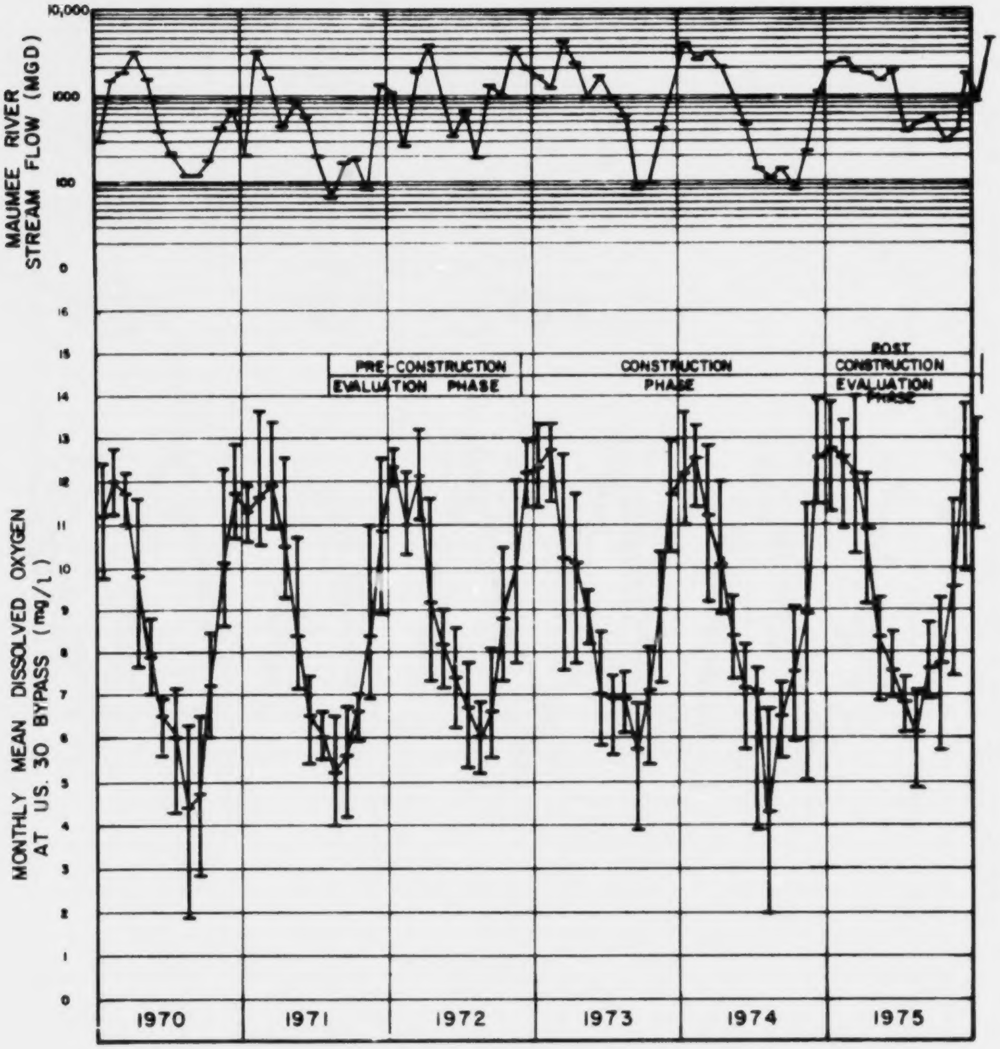


FIGURE 16 - MAUMEE RIVER MEAN STREAM DISSOLVED OXYGEN
U. S. 30 BYPASS BRIDGE
JANUARY, 1970 TO FEBRUARY, 1976

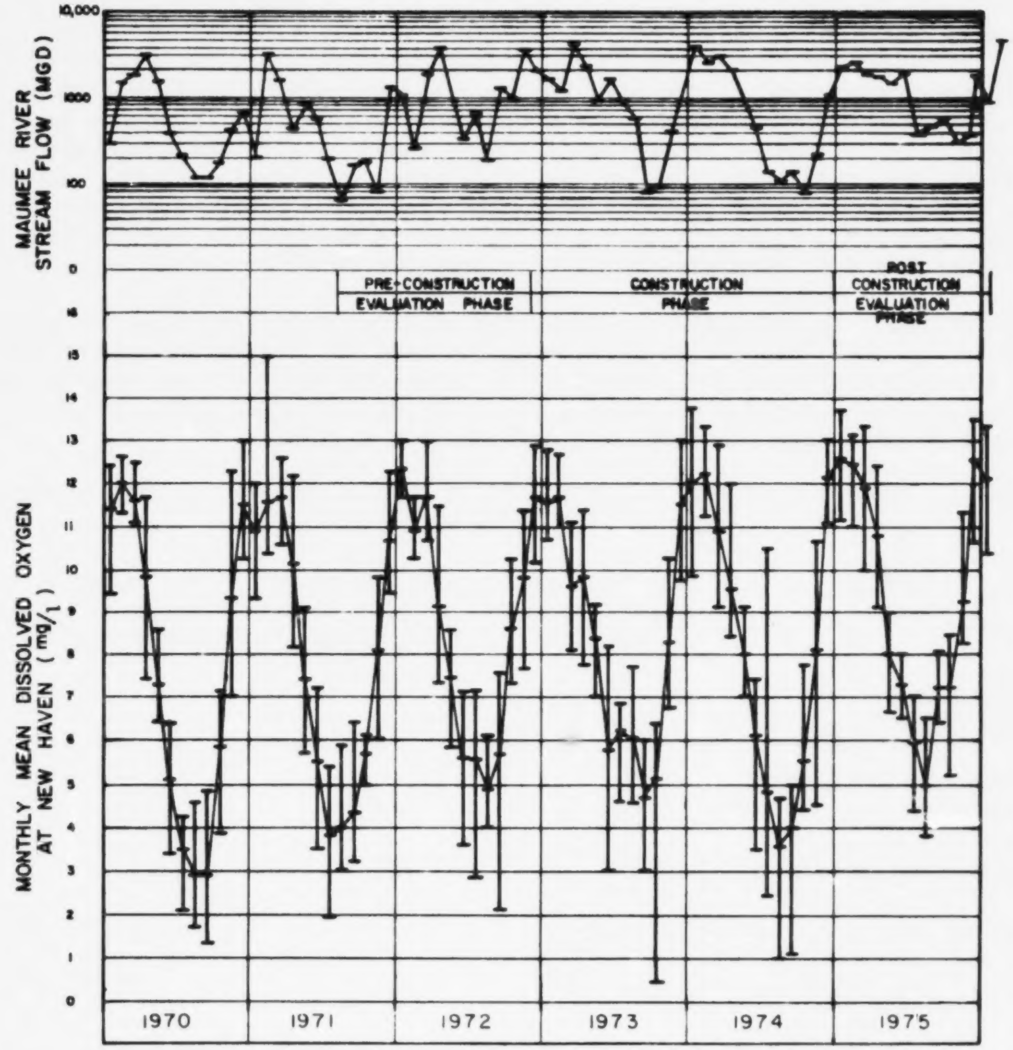


FIGURE 17 - MAUMEE RIVER MEAN STREAM DISSOLVED OXYGEN
NEW HAVEN BRIDGE AT LANDIN ROAD
JANUARY, 1970 TO FEBRUARY, 1976

Only one-year's data are presented for the post-construction evaluation period. However, it should be apparent that there has been a significant reduction in both the frequency of occurrence of low stream D.O. and the minimum D.O. observed. Future data-gathering should determine whether this trend is a permanent result of the diversion of some of the raw stormwater-induced combined sewer overflows to treatment.

Biochemical Oxygen Demand

Pre-construction sampling results for average stream BOD_5 increased downstream from the pond effluent to the U.S. 30 Bypass Bridge, as shown in Table 6. The New Haven Bridge samples were found to have decreased slightly, implying some stream recovery on the utilization of organic matter. However, this decreased exertion probably cannot be attributed to any property of the stream recovery because the amount of difference between the samples taken at New Haven and U.S. 30 Bypass could be attributed to experimental error alone.

Furthermore, this indication of some kind of stream recovery does not carry over to the dissolved oxygen data described earlier. It is concluded that the decrease in stream BOD_5 from U.S. 30 Bypass to the New Haven Bridge during the pre-construction evaluation is an anomaly of these data and not an indication of stream recovery over this short a distance.

Table 6. PRE-CONSTRUCTION EVALUATION PHASE STREAM BIOCHEMICAL OXYGEN DEMAND
August, 1971 to November, 1972

	Anthony Boulevard (1/4 mile upstream)	U.S. 30 Bypass (1-1/2 miles downstream)	New Haven (4-1/2 miles downstream)
Arithmetic Mean Stream BOD_5 (mg/l)	4.29	6.85	5.51
Standard Deviation (mg/l)	2.82	4.95	3.31
Number of Observations	441	441	443
Range of Values	0.2 to 27.	0.5 to 27.	0.5 to 25.

Note that the range between maximum and minimum for each sampling site is basically the same for all. The maximum stream BOD_5 is 27 milligrams per liter.

Post-construction evaluation phase data taken at the same three sampling points show essentially the same trend as the pre-construction phase, considering a range of probable sampling and analysis error of ±20 percent. These data are presented in Table 7. The range of values between maximum and minimum for each location has markedly decreased in these data. This decrease in range also resulted in a decrease in maximum stream BOD_5. This result is interpreted as a manifestation of stream improvements that may be partly attributed to combined sewer stormwater overflow treatment at the Water Pollution Control Plant. It tends to confirm the results presented in the section discussing stream D.O. where a noticeable increase in the stream mean D.O. was noted between pre-construction and post-construction periods.

Table 7. POST-CONSTRUCTION EVALUATION PHASE STREAM BIOCHEMICAL OXYGEN DEMAND
January, 1975 to February, 1976

	Anthony Boulevard (1/4 mile upstream)	U.S. 30 Bypass (1-1/2 miles downstream)	New Haven (4-1/2 miles downstream)
Arithmetic Mean Stream BOD_5 (mg/l)	2.01	3.95	4.34
Standard Deviation (mg/l)	3.21	2.05	2.27
Number of Observations	201	190	191
Range of Values Observed (mg/l)	0.1 to 11.0	0.1 to 11.0	0.2 to 11.0

Maumee River chemical oxygen demand samples taken at these same three points show roughly some of these same trends. However, there are fewer data in the post-construction evaluation available for comparison.

As a result, no conclusions were drawn from these data compared to the pre-construction phase.

Fecal Coliform Group Indicator Organisms

Bacteriological indicators were monitored in the stream sampling program in both the pre-construction and post-construction evaluation work. The same three sampling points were monitored in both evaluation periods. Upstream at Anthony Boulevard (1/4 mile upstream), downstream at U.S. 30 Bypass (1-1/2 miles downstream), and at New Haven Bridge (4-1/2 miles downstream). Table 8 shows the summary results of the pre-construction sampling program for fecal coliform group indicator organisms.

Table 8. PRE-CONSTRUCTION EVALUATION PHASE STREAM FECAL COLIFORM GROUP INDICATOR ORGANISMS
August, 1971 to November, 1972

	Anthony Boulevard (1/4 mile upstream)	U.S. 30 Bypass (1-1/2 miles downstream)	New Haven (4-1/2 miles downstream)
Geometric Mean Fecal Coliform Group (N/100 ml)	1,650	9,000	4,800
Standard Deviation (N/100 ml)	116,000	392,000	476,000
Number of Observations	322	340	345
Range of Values Observed (N/100 ml)	0 to 29,000	0 to 1,400,000	0 to 6,400,000

It should be noted that there appears to be a bacterial "die-off" phenomenon at work from the U.S. 30 Bypass to the New Haven Bridge sampling points. With normal stream transit times of from 0.5 to perhaps 5 hours between these two sampling points, this result may represent a declining population phase for this class of organisms.

Total coliform indicator organisms sampled in the pre-construction phase showed similar trends to the fecal coliform group. These data were not tabulated because of insufficient numbers of events analyzed to statistically evaluate.

Fecal coliform data were selected in the post-construction period as the most reliable indicator of the bacteriological contamination of the stream from combined sewer overflows. Table 9 shows the comparable results from the post-construction evaluation phase.

From the examination of the geometric mean distributions of the data for fecal coliform group indicator organisms, it should be apparent that there has been a significant improvement in the stream water quality as measured by the fecal coliform MF procedure. Figure 18 shows these data plotted on log normal coordinates for the pre-construction evaluation phase. Figure 19 similarly displays the data for the post-construction evaluation phase.

As shown in Figure 19, it appears as though stream background fecal coliform measured at Anthony Boulevard must be decreased before any further improvement in water quality downstream from the plant can be achieved.

Table 9. POST-CONSTRUCTION EVALUATION PHASE STREAM FECAL COLIFORM GROUP INDICATOR ORGANISMS
January, 1975 to February, 1976

	Anthony Boulevard (1/4 mile upstream)	U.S. 30 Bypass (1-1/2 miles downstream)	New Haven (4-1/2 miles downstream)
Geometric Mean Fecal Coliform Group (N/100 ml)	920	1,000	800
Standard Deviation (N/100 ml)	63,000	44,000	78,000
Number of Observations	131	125	125
Range of Values Observed (N/100 ml)	0 to 80,000	0 to 150,000	0 to 5,000,000

Total Phosphorus

Stream water quality measurements of total phosphorus have been regularly taken by Water Pollution Control Plant personnel for several years. This program included sampling and analysis of the Maumee River at the same three points for total phosphorus during and after each stormwater event. The data obtained for stream total phosphorus in the pre-construction program are shown in Table 10 and the corresponding post-construction data are given in Table 11.

Table 10. PRE-CONSTRUCTION EVALUATION PHASE STREAM TOTAL PHOSPHORUS
August, 1971 to November, 1972

	Anthony Boulevard (1/4 mile upstream)	U.S. 30 Bypass (1-1/2 miles downstream)	New Haven (4-1/2 miles downstream)
Geometric Mean* Total Phosphorus (mg/1)	0.37	0.78	0.90
Standard Deviation (mg/1)	0.21	0.90	0.72
Number of Observations	423	427	413
Range of Values Observed (mg/1)	0.1 to 2.0	0.1 to 3.5	0.1 to 3.4

*Log normal distributions provided the best fit of these data.

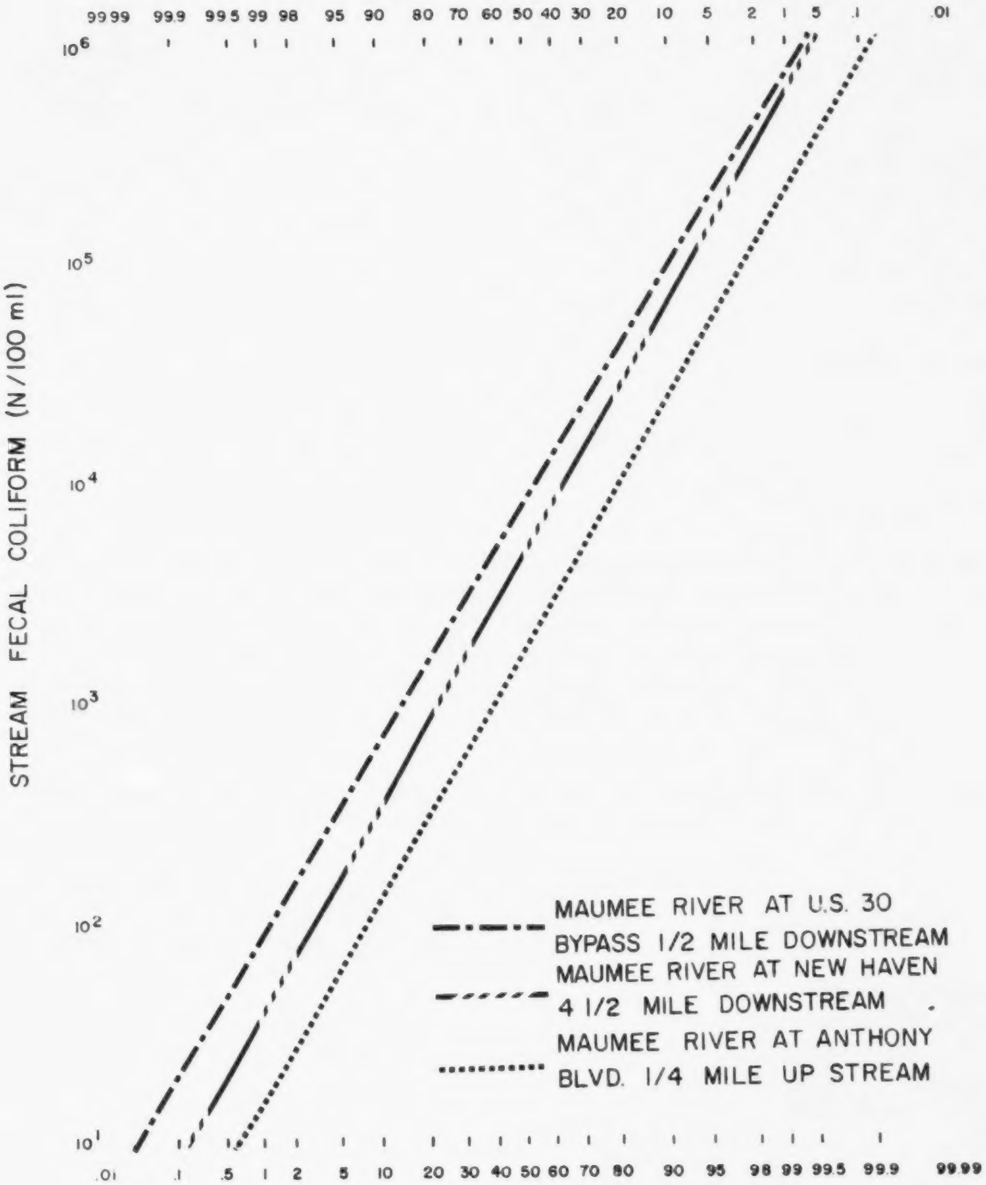


FIGURE 18 - STREAM FECAL COLIFORM INDICATOR ORGANISMS
PRE-CONSTRUCTION EVALUATION PHASE
AUGUST, 1971 TO NOVEMBER, 1972

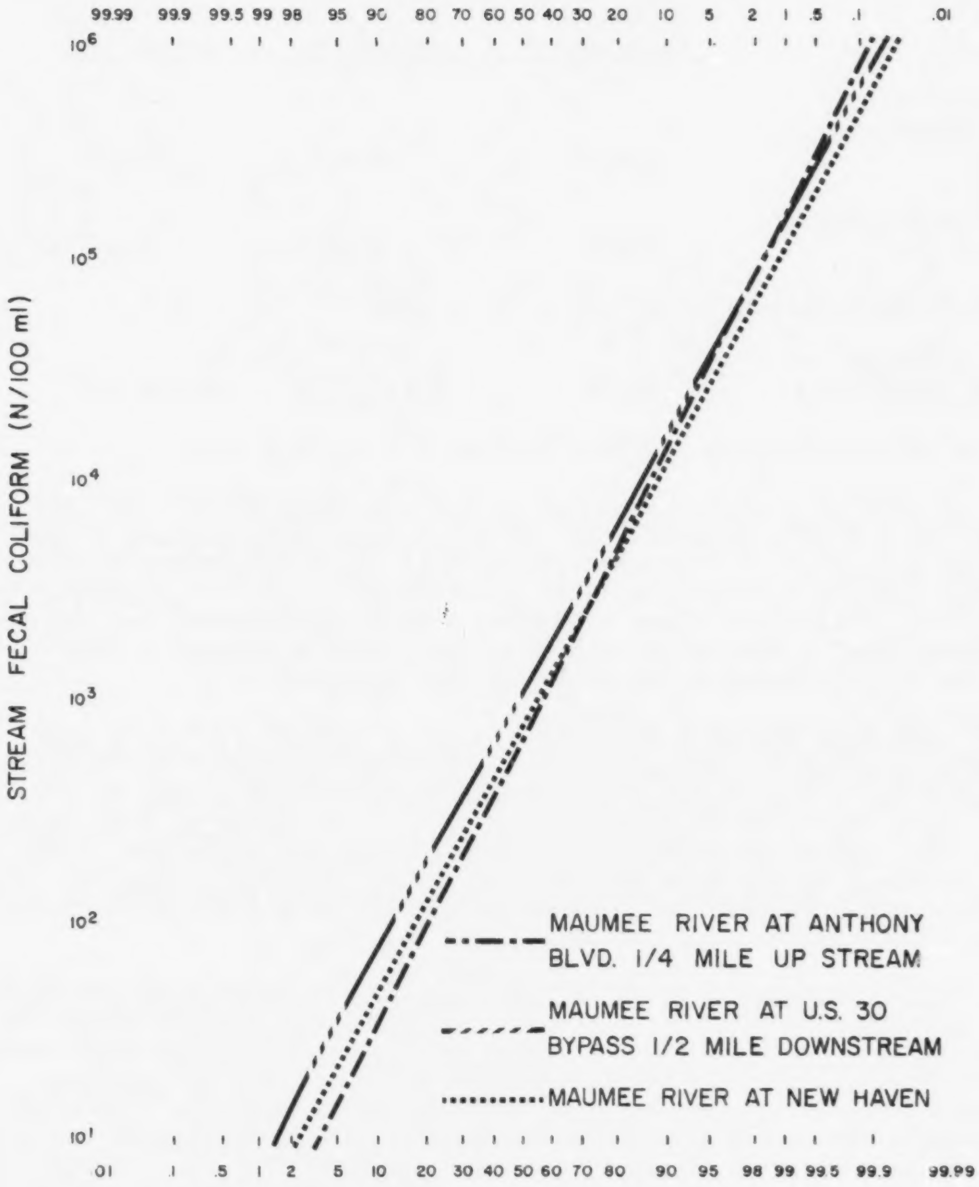


FIGURE 19 - STREAM FECAL COLIFORM GROUP INDICATOR ORGANISMS
POST-CONSTRUCTION EVALUATION PHASE
JANUARY, 1975 TO FEBRUARY, 1976

Table 11. POST-CONSTRUCTION EVALUATION PHASE STREAM TOTAL PHOSPHORUS
January, 1975 to February, 1976

	Anthony Boulevard (1/4 mile upstream)	U.S. 30 Bypass (1-1/2 miles downstream)	New Haven (4-1/2 miles downstream)
Geometric Mean* Total Phosphorus (mg/l)	0.40	0.37	0.40
Standard Deviation (mg/l)	0.25	0.32	0.31
Number of Observations	174	139	143
Range of Values Observed (mg/l)	0.1 to 3.1	0.1 to 1.7	0.1 to 2.3

*Log normal distributions provided the best fit of these data.

There are several plausible explanations for the reduction in stream total phosphorus from 1971-72 to 1975-76. First, a State-wide detergent phosphate law went into effect in Indiana early in 1971. This legislation limited household laundry detergents sold in the State to 8.7 percent as phosphorus until 1973. Thereafter, these detergents sold in Indiana were required to be "phosphate-free". Some of the change in the stream water quality at Fort Wayne may be attributed to the effect of this legislation.

Another plausible alternate hypothesis to the diversion, treatment, and detention of combined sewer overflows as the explanation for the improved water quality as measured by total phosphorus lies in the manner of operation of the Water Pollution Control Plant itself. In the summer of 1973, this source was required by Indiana Stream Pollution Control Board regulations to begin treatment of the plant effluent for removal of total phosphorus. The results of this treatment is shown in the discussion on plant effluent data taken at the terminal pond discharge.

These results may also be noted as showing that the post-construction evaluation data given in Table 11 show essentially the same trend of values upstream and downstream from the treatment plant and the combined sewer overflows. This result may be significant: it indicates that no additional treatment of municipal wastewater at whatever origin will result in a further reduction in the stream pollutional levels of total phosphorus, if the data taken at the Anthony Boulevard Bridge truly represent the stream background condition with respect to this pollutant. Additional upstream data-gathering outside the scope of this work will be necessary to substantiate this conclusion.

Figure 54, following, depicts the long-term plant effluent total phosphorus levels have markedly decreased since the phosphorus removal facility start-up in July, 1973. This result tends to substantiate the conclusion that the stream water quality improvement in total phosphorus is mostly attributable to phosphorus removal at the Water Pollution Control Plant.

Ammonia Nitrogen

Stream water quality monitoring in the pre-construction evaluation did not include sufficient data for ammonia nitrogen to show any significant trends. In the post-construction evaluation phase, these data were periodically taken in dry weather as well as during storm events. The results seem to fit a log normal distribution better than a normal distribution. Table 12 shows the results for the distribution that best fit the data taken.

Table 12. POST-CONSTRUCTION EVALUATION PHASE AMMONIA NITROGEN
January, 1975 to February, 1976

	Anthony Boulevard (1/4 mile upstream)	U.S. 30 Bypass (1-1/2 miles downstream)	New Haven (4-1/2 miles downstream)
Geometric Mean Ammonia Nitrogen (mg/l)	0.42	0.84	0.90
Standard Deviation (mg/l)	0.50	0.63	0.84
Number of Observations	134	135	140
Range of Values Observed (mg/l)	0.05 to 2.0	0.2 to 4.0	0.1 to 4.0

Suspended Solids (Nonfilterable Residue)

Pre-construction and post-construction sampling of stream suspended solids (nonfilterable residue) showed virtually no change in stream water quality that could be attributed to the stormwater treatment program. The distributions obtained are tabulated in statistical summary form in Table 13.

Volatile suspended solids data gathered in the pre-construction evaluation essentially showed a bimodal distribution depending on the season. Cold-weather months seemed to have greater frequencies of low volatile data (10 to 20 percent VSS) while in contrast, warmer weather data showed a tendency to cluster in the range of 25 to 35 percent VSS. This result is not considered particularly significant because the values of VSS are dependent on the values of total suspended solids. The data for total suspended solids gathered in the preconstruction phase essentially showed no difference in this parameter. There were too few data gatherings in the post-construction evaluation stream sampling to warrant any comparisons for pre-construction to post-construction for volatile suspended solids.

Table 13. STREAM SUSPENDED SOLIDS
Nonfilterable Residue

	Anthony Boulevard (1/4 mile upstream)		U.S. 30 Bypass (1-1/2 miles downstream)		New Haven (4-1/2 miles downstream)	
	Pre-C.	Post-C.	Pre-C.	Post-C.	Pre-C.	Post-C.
Geometric Mean Suspended Solids (mg/1)	60	60	72	72	70	105
Standard Deviation	195	315	155	190	150	125
Number of Observations	266	189	257	191	272	195
Range of Values Observed (mg/1)	5 to 1,390	5 to 1,600	5 to 1,400	5 to 1,100	5 to 800	5 to 1,300

Note: The pre-construction evaluation period included August, 1971 through November, 1972. The post-construction evaluation period began January 1, 1975 and ended February 6, 1976.

Stormwater Facility Evaluation Parameters

The principal Stormwater Facility parameters, which formed the basis of the comparative evaluation for the treatment equipment used in this program, were settleable and flotable coarse solids and hydraulic capacity.

Settleable and Flotable Debris--

A distinguishing characteristic of raw, untreated combined sewer storm flows received at the Fort Wayne Water Pollution Control Plant has been the amount of coarse solids and debris contained in the storm flow. These materials have been screened continuously at the plant by a mechanical bar screen for years, resulting in very large quantities of screened materials necessitating sanitary disposal. Since the stormwater station was designed to handle storm-caused overflows which would have gone to the river without treatment, a trash rack with bar spacing of 2-1/4 inches was provided.

Figure 20 shows the nature of the screened materials that have been deposited on the trash rack. In one location (not shown), a beer can was found firmly lodged between the vertical bars in an upright position. Plant operating personnel report that extraction of these materials using the motorized bar rack mechanism has been mostly unsuccessful because these materials are light and easily dislodged when dry, and regularly pass through the bar spacing.

Figure 21 is a photograph of the fibrous, stringy debris hanging from the horizontal members on the trash rack taken from a boat in the final weeks of evaluation. This type of debris was also noticeable in the chlorine contact channel after long, sustained events where the raw stormwater pumps operated for a number of hours.

One of the reasons for the incursion by boat into the Stormwater Facility wetwell was to inspect the condition of the wetwell and raw stormwater pumps. Figure 22 is a photograph taken on December 8, 1975, in the wetwell at a depth of 9 feet on the depth indicator.

The pump shown is the Johnston 30 PS pump. There was no evidence of external wear or damage resulting from pumping raw combined sewer overflows.

Table 14 presents the results of wetwell soundings and sampling results taken on December 8, 1975, after the facility had been in service for nearly one year. The depth of sediment measured in this survey resulted from standing water of depth from 7 to 11 feet (2.14 to 3.35 meters) in the wetwell and river crossing conduits. All solids data are for total sample volume (about one liter) except Sample No. 1B which is sediment only. All depths were taken by sounding with a graduated survey level rod and bottom sampling tube.

Table 14. STORMWATER FACILITY WETWELL SEDIMENTS
SAMPLING SURVEY RESULTS
December 8, 1975

Sample No.	Location	Depth of Sediments (feet)	Sediment Specific Gravity	Total Solids (Percent)	Volatile Solids (Percent)
1A	C_L 25 MGD Pump & C_L Wetwell 25' N of Trash Rack	1.5	1.002	4.4	27.0
1B	10' R+ & 10' N of C_L 25 MGD Pump	1.5	**	31.9	31.0
2A	NE Corner of Wetwell	0.5	1.062	13.7	26.0
2B	NW Corner of Wetwell	0.25	1.016	2.9	49.0
3A	Center Bay 25' S of N Wall	0.5	1.026	3.1	29.0
3B	W Side 35' S of N Wall	0.5	1.020	2.5	31.0

**Insufficient Sample Volume.

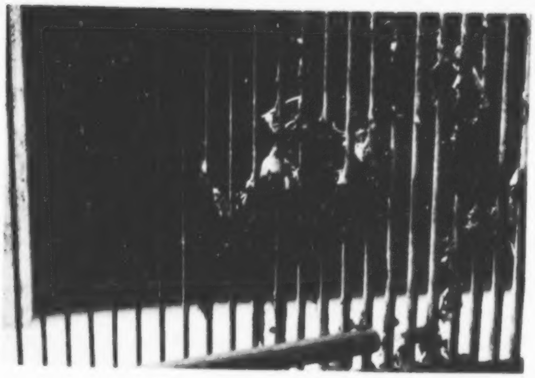


Figure 20 - Screened Material Adhering to Trash Rack Vertical Bars

Figure 21 - Interior View of Stormwater Facility Trash Rack Showing Stringy Debris

Figure 22 - Johnston 30 PS Pump Column and Suction Bell After Eleven Months of Operation

Given the highly variable nature of the combined sewer overflows, it seemed logical to assume that any removal of the gross solids would benefit the screening devices. The debris shown in Figures 20 and 21 would likely cause problems downstream if not removed somehow. Figure 23 is a photograph of the chlorine contact channel scum removal pipe resulting from an event on July 19, 1975. This material normally was removed in screening unless the head tank or Rex headbox overflows allowed it to bypass screening by overflowing to the effluent channel.

Among the screening methods studied during this program, the Bauer Hydrasieve® appeared to best deal with gross solids debris. The Envirex Rotary Drum Screen and SWECO Centrifugal Wastewater Concentrator produce more liquified concentrate that required further treatment at the Water Pollution Control Plant. In contrast to this, the Bauer produced a semidry solid residue that was usually hosed into a 55-gallon drum and trucked off to a sanitary landfill for ultimate disposal. This method of disposal is preferred for stringy debris because these materials are less putrescible than other organic solids. Furthermore, this debris sometimes is not caught in the WPCP moving bar screen and winds up in the plant raw sludge feed to the anaerobic digesters. Stringy nondegradable debris has been a problem in the anaerobic digesters at Fort Wayne in the past.

Figures 24 and 25 show photographs of the drying debris strained from the storm flow. Note the plastic materials such as plastic bandages, plastic drinking straw, and magnetic recording tape (upper left corner, Figure 25). These materials are known to be nonbiodegradable using conventional sludge handling and disposal treatment techniques. Once removed from the wastewater by screening, it is possible to dispose of these dry solids at a sanitary landfill. However, to operate these screens in this manner reduces the unit effective hydraulic loading somewhat.

Figure 26 shows Bauer screenings collection channel after an event where the unit hydraulic loading was limited to approximately three-fourths of design flow to produce the semidry solids shown in Figures 24 and 25. This corresponds to a surface hydraulic loading rate of approximately 25 gpm/sq. ft., or 1,050 lpm/m^2.

At full hydraulic design capacity of 200 gpm per foot (2,500 lpm per meter) of screen width (35 gpm/sq. ft., or 1,400 lpm/m^2), the photograph of this center channel shows the wastewater "skipping" along the surface of the screens (Figure 27). Note that the unit at full capacity does produce somewhat more flow in the screening concentrate channels while giving up the desirable characteristic of semidry solids screenings.

Hydraulic Capacity--

One of the consequences of operating a large scale facility for the treatment of combined sewer overflows is that the facility must be capable of taking everything that "comes down the pipe", so to speak. This necessitates some compromises within the physical constraints of the system. The Stormwater Facility at Fort Wayne was designed to pump and screen 75 MGD (3.286 cumec). However, there were times when the nominal capacity of the screening equipment

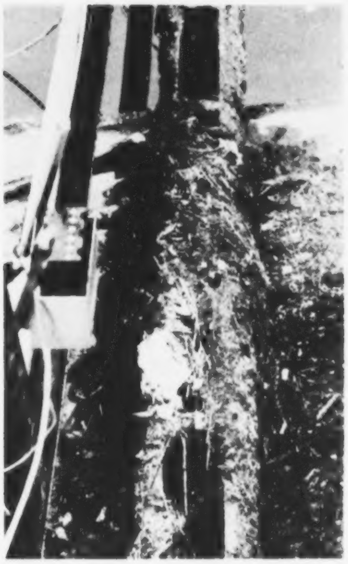


Figure 23 - Chlorine Contact Channel Flotable Debris Collected at Scum Removal Pipe - July 24, 1975

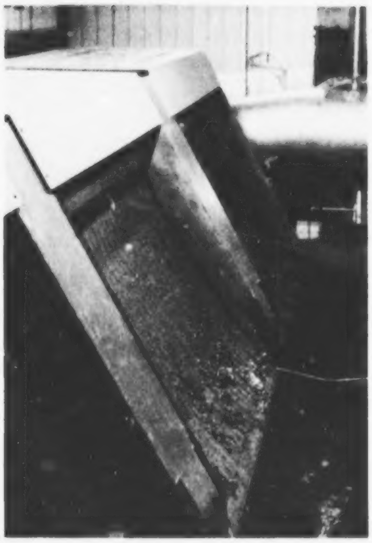


Figure 24 - Bauer Hydrasieve® Screenings After Stormwater Event

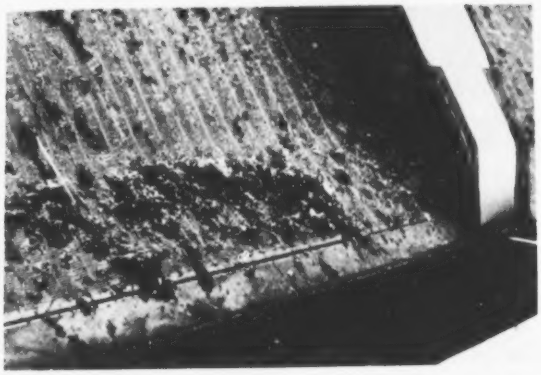


Figure 25 - Close-Up of Semidry Screenings Collected During Stormwater Event From Bauer Hydrasieve®

Figure 26 - Bauer Hydrasieve® Center Concentrate Channel Showing Semidry Screenings Collected

Figure 27 - Bauer Hydrasieve® Operating at Full Hydraulic Capacity

was inadequate to treat all the wastewater received at the facility. In order to minimize the amount of storm-caused overflows to the river, the decision was made to pump as much of the flow to the screening room head tank as possible and allow it to flow by gravity head either to the screening units or to overflow to the screened effluent channel. In addition to this raw unscreened waste overflow point, the Rex screen headbox has an overflow point that leads to the screened effluent channel also (See Figure 28).

The overflows from the head tank were not metered in this program. The overflows from the Rex Rotary Drum Screen headbox were recorded by a circular chart stage recorder. Regardless of whether stormwater from these two overflow bypasses in the facility received any type of screening, these wastes were pumped to the elevation of the chlorine contact channel where they were disinfected and run through the full detention pond period. The pond is an inherent part of the treatment scheme and it should be recognized that a high degree of treatment efficiency can usually be expected from such a pond.

The Rex Rotary Drum Screen overflow shown in Figure 28 is recirculated pond water, and it appears in the photograph as clear and free of noticeable solids. However, more typically, raw storm flow contained coarse solids and all of the raw waste pollution. The chlorine contact channel is more than 1,100 feet (335 meters) long. In this length, the velocity of flow is purposely less than 1.5 feet per second (0.46 meter per second) to avoid scouring the bottom and undermining the fill material that makes up the levees around Terminal Pond No. 1. A 3 mil polyethylene channel liner was torn in many places in the first few weeks of operation of the facility and did not serve any useful purpose thereafter.

Figure 29 shows a longitudinal view of the channel after eleven months of operation. Note the sludge banks near the scum baffle. Figure 30 is another view of this same area. These photographs are typical of the channel bottom along its length.

Similar sludge deposits formerly could be seen along the Maumee River at low water. Since these deposits now occur at the facility rather than the stream some method of treatment can be used to neutralize and/or remove these settled solids.

The sediment deposits shown in Figures 29 and 30 are expected to continue throughout the useful lifetime of the facility. These sediments also have been observed to a depth of 1.5 feet (0.46 meter) in the terminal pond. A small self-contained dredge has been operating in the terminal pond subsequent to the termination of the evaluation phase, beginning in March, 1976. This dredging of these sediments has been responsible for removal of most of these deposits in the pond. However, the dredge unit cannot be used in the chlorine channel itself because its draft is more than the channel depth.

A good technique for cleaning up these sediments would be to sluice the settled solids with a high pressure water hose. However, the plastic channel liner has deteriorated to the extent that any extensive amount of water sluicing would result in excessive erosion of the channel banks and levees. It is recommended that future channels designed for this purpose be lined

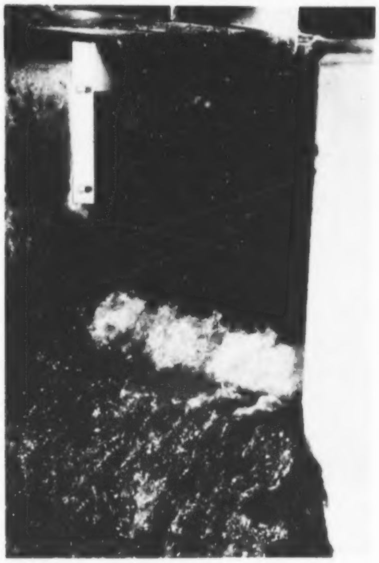


Figure 28 - Rex Rotary Drum Screen Headbox Overflow Point

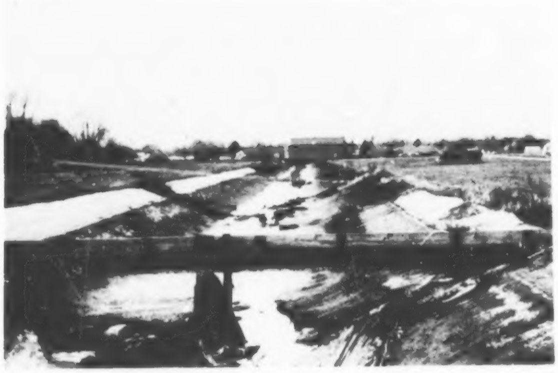


Figure 29 - Chlorine Contact Channel From Overflow Weir to Terminal Pond Showing Sludge Bank Deposits

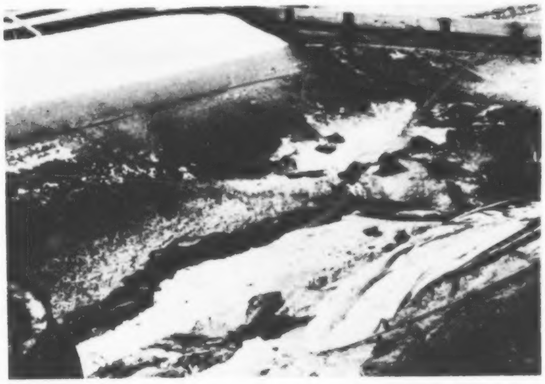


Figure 30 - Chlorine Contact Channel Bottom at Overflow Weir to Terminal Pond

with impervious bituminous or Portland cement concrete paving material with underdrains to carry the sluiced sediments to the ultimate disposal point.

Stormwater Screening Equipment Comparative Evaluation

Table 15 presents the screening equipment comparative evaluation on the basis of overall treatment efficiency for suspended solids, BOD_5, COD, total phosphorus, and ammonia nitrogen (NH_4^+-N). Note that there were negative treatment efficiencies determined for each of the screening methods considered in this program.

Negative treatment efficiencies could be misinterpreted without some explanation. Measurement error in either flow or concentrations may be responsible for some of the negative efficiency parameters. Also, the time of sample taking may account for some of the variation noted. The few number of events (as few as 25 events for volatile suspended solids sampling) could also account for some of the variation noted. The fluid shear observed in the screening methods themselves may account for a portion of the variation and negative results.

The manner of calculation of the screening equipment efficiencies was programmed by computer to perform the computations of loadings by increments for which there were sufficient concentration data taken. These incremental loadings were then summed to obtain the sampling location event loadings. The sampling location event loadings were then used to compute the treatment equipment efficiencies. For the sake of relative comparisons, the negative efficiency results so obtained are recorded in Table 15 as zero efficiency with an asterisk (*) superscript to denote negative results obtained.

Suspended Solids (Nonfilterable Residue)--

Figure 31 is a graphical "scattergram" of the relationship between the three screening equipment treatment efficiencies as a function of each screening method rated hydraulic loading.* This approach was selected as the common determinant among the different screening methods evaluated because it relates directly to the hydraulic flux rate for the rotating screening devices (i.e., Rex and SWECO units) operated at a constant rotational speed as well as the fixed screening area Bauer Hydrasieve®. The screen drive for the Rex unit was maintained at a constant rate of rotation throughout most of this program and the variation in flux rate was obtained by varying the raw stormwater flow.

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*Rated hydraulic loading for the screens evaluated was based on the manufacturer's recommended hydraulic loading rate as determined in the equipment bidding procedure. The nominal rated hydraulic loading for the Bauer Hydrasieve® was 13,125 U.S. gpm (49,700 liters per minute), for the Envirex Rotary Drum Screen was 12,600 U.S. gpm (47,700 liters per minute), and for each SWECO CWC®, the rated loading was 3,100 U.S. gpm (11,700 liters per minute).

TABLE 15. SCREENING EQUIPMENT COMPARATIVE EVALUATION OF OVERALL TREATMENT EFFICIENCY

Event No.	Raw Waste Source	Screening Duration	Screening Units	Overall Treatment Efficiency in Percent					Total Flow Treated in MG
				Suspended Solids	Biochemical Oxygen Demand	Chemical Oxygen Demand	Total Phosphorus	Ammonia Nitrogen	
148	Stormwater	3h.0m.	Bauer	54	9	24	9	--	0.63
			Rex	61	4	12	23	--	1.53
			SWECO	54	9	29	8	--	0.90
159	Stormwater	7h.50m.	Bauer	52	0*	0*	40	--	7.20
			Rex	59	0*	0*	43	--	0.28
			SWECO	40	0*	0*	45	--	7.09
165	Stormwater	0h.55m.	Bauer	56	60	62	--	--	0.90
			Rex	45	70	54	--	--	0.90
			SWECO	50	31	55	--	--	0.90
166	Stormwater	2h.40m.	Bauer	17	0*	12	--	--	1.65
			Rex	2	0*	0*	--	--	1.60
			SWECO	0*	0*	0*	--	--	1.98
167	Stormwater	2h.35m.	Bauer	29	0*	0	6	--	1.55
			Rex	27	0*	1	0*	--	1.55
			SWECO	9	0*	0*	0*	--	1.55
170	Stormwater	4h.55m.	Bauer	44	27	31	13	--	2.50
			Rex	--	--	--	--	--	--
			SWECO	48	38	22	0*	--	6.1

TABLE 15. SCREENING EQUIPMENT COMPARATIVE EVALUATION OF OVERALL TREATMENT EFFICIENCY - Continued

Event No.	Raw Waste Source	Screening Duration	Screening Units	Overall Treatment Efficiency in Percent: Suspended Solids	Overall Treatment Efficiency in Percent: Biochemical Oxygen Demand	Overall Treatment Efficiency in Percent: Chemical Oxygen Demand	Overall Treatment Efficiency in Percent: Total Phosphorus	Overall Treatment Efficiency in Percent: Ammonia Nitrogen	Total Flow Treated in MG
173	Stormwater	2h.40m.	Bauer	0	0	37	0*	--	1.65
			Rex	--	--	--	--	--	--
			SWECO	0*	0*	14	0*	--	4.00
181	Stormwater	1h.05m.	Bauer	--	--	--	--	--	--
			Rex	40	25	37	7	--	0.97
			SWECO	79	74	77	50	--	0.20
182	Stormwater	4h.15m.	Bauer	17	4	16	53	--	0.53
			Rex	15	1	7	0*	--	2.73
			SWECO	0*	0*	0*	0*	--	2.18
185	Stormwater	2h.52m.	Bauer	--	--	--	--	--	--
			Rex	17	27	0*	0*	--	1.85
			SWECO	66	74	58	40	--	2.00
186	Stormwater	4h.0m.	Bauer	0*	62	0*	--	--	0.72
			Rex	0*	0*	0*	0*	0*	4.65
			SWECO	5	7	0*	0*	0*	1.76
187	Stormwater	6h.55m.	Bauer	0*	14	1	0*	25	2.88
			Rex	11	11	7	10	5	4.38
			SWECO	62	41	42	50	43	1.75

TABLE 15. SCREENING EQUIPMENT COMPARATIVE EVALUATION OF OVERALL TREATMENT EFFICIENCY - Continued

Event No.	Raw Waste Source	Screening Duration	Screening Units	Overall Treatment Efficiency in Percent					Total Flow Treated in MG
				Suspended Solids	Biochemical Oxygen Demand	Chemical Oxygen Demand	Total Phosphorus	Ammonia Nitrogen	
191	Stormwater	3h.30m.	Bauer	0*	0*	0*	0*	0*	0.84
			Rex	1	-	0*	13	9	3.22
			SWECO	28	10	14	43	25	1.98
192	Stormwater	12h.50m.	Bauer	0*	0*	0*	0*	0*	5.49
			Rex	0*	0*	0*	0*	0*	10.17
			SWECO	0*	0*	0*	0*	1	8.82
193	Stormwater	4h.15m.	Bauer	4	8	3	0*	4	3.26
			Rex	24	0*	0*	0*	0*	2.81
			SWECO	26	3	14	5	9	5.40
194	Pond Recirculation	1h.15m.	Bauer	0*	11	23	13	16	1.40
			Rex	0*	57	24	14	0	1.59
			SWECO	20	19	33	54	27	0.09
195	Stormwater	4h.0m.	Bauer	0*	0	0*	0*	0*	2.66
			Rex	4	28	2	14	17	1.92
			SWECO	15	13	15	0*	3	3.59
197	Stormwater	2h.10m.	Bauer	28	48	34	46	--	1.52
			Rex	54	75	62	50	--	0.56
			SWECO	54	21	18	48	--	1.90

TABLE 15. SCREENING EQUIPMENT COMPARATIVE EVALUATION OF OVERALL TREATMENT EFFICIENCY - Continued

Event No.	Raw Waste Source	Screening Duration	Screening Units	Overall Treatment Efficiency in Percent					Total Flow Treated in MG
				Suspended Solids	Biochemical Oxygen Demand	Chemical Oxygen Demand	Total Phosphorus	Ammonia Nitrogen	
198	Stormwater	6h.50m.	Bauer	1	0*	5	23	40	5.14
			Rex	12	0*	21	32	42	2.31
			SWECO	27	0*	45	42	49	5.59
200	Stormwater	1h.15m.	Bauer	29	16	31	23	8	1.15
			Rex	7	0*	0*	32	0*	0.46
			SWECO	42	25	26	36	22	1.08
203	Stormwater	6h.50m.	Bauer	0*	21	9	12	6	5.04
			Rex	0*	7	3	0	10	1.86
			SWECO	19	45	30	0*	24	6.53
207	Raw Stormwater	3h.40m.	Bauer West	24	17	24	28	16	1.12
			Rex	--	--	--	--	--	--
			SWECO	35	30	28	31	8	1.91
207T**	Tandem Run	3h.40m.	Bauer East	24	17	24	28	16	1.12
			Rex	0*	7	31	0	0	0.41
			SWECO	--	--	--	--	--	--
212	Stormwater	2h.40m.	Bauer	0*	--	0*	0*	9	0.77
			Rex	5	--	0*	0*	6	0.54
			SWECO	0*	--	0*	0*	33	1.26

TABLE 15. SCREENING EQUIPMENT COMPARATIVE EVALUATION OF OVERALL TREATMENT EFFICIENCY - Continued

Event No.	Raw Waste Source	Screening Duration	Screening Units	Overall Treatment Efficiency in Percent					Total Flow Treated in MG
				Suspended Solids	Biochemical Oxygen Demand	Chemical Oxygen Demand	Total Phosphorus	Ammonia Nitrogen	
213T**	Tandem Run on Stormwater	1h.45m.	Bauer	32	27	27	26	0*	
			Rex	23	33	29	32	30	0.28
			SWECO	14	6	13	3	68	1.25
214T**	Tandem Run on WPCP Raw Sewage		Bauer	5	0*	0*	0*	0	1.72
			Rex	0*	0*	0*	4	0*	
			SWECO	32	0*	9	0*	26	0.45
215T**	Tandem Run on WPCP Raw Sewage		Bauer	0*	0*	2	0*	0*	2.42
			Rex	0*	0*	0*	0*	0*	0.54
			SWECO	0*	0*	0*	0*	0*	0.54
216T**	Tandem Run on WPCP Raw Sewage		Bauer	8	0*	0*	0*	0*	1.60
			Rex	5	0*	4	0*	0*	0.33
			SWECO	5	0*	5	0*	0*	0.38
217T**	Tandem Run on WPCP Raw Sewage		Bauer East	0*	0*	0*	0*	0*	1.70
			Rex	0*	0*	0*	0*	0*	0.51
			SWECO	14	0*	0*	36	0*	0.41

* Negative treatment efficiency results obtained in raw data are adjusted to zero.

** Tandem runs were performed with Bauer East as prescreening device ahead of Rex and SWECO North bank units.

Note: MG x 3.785 = million liters

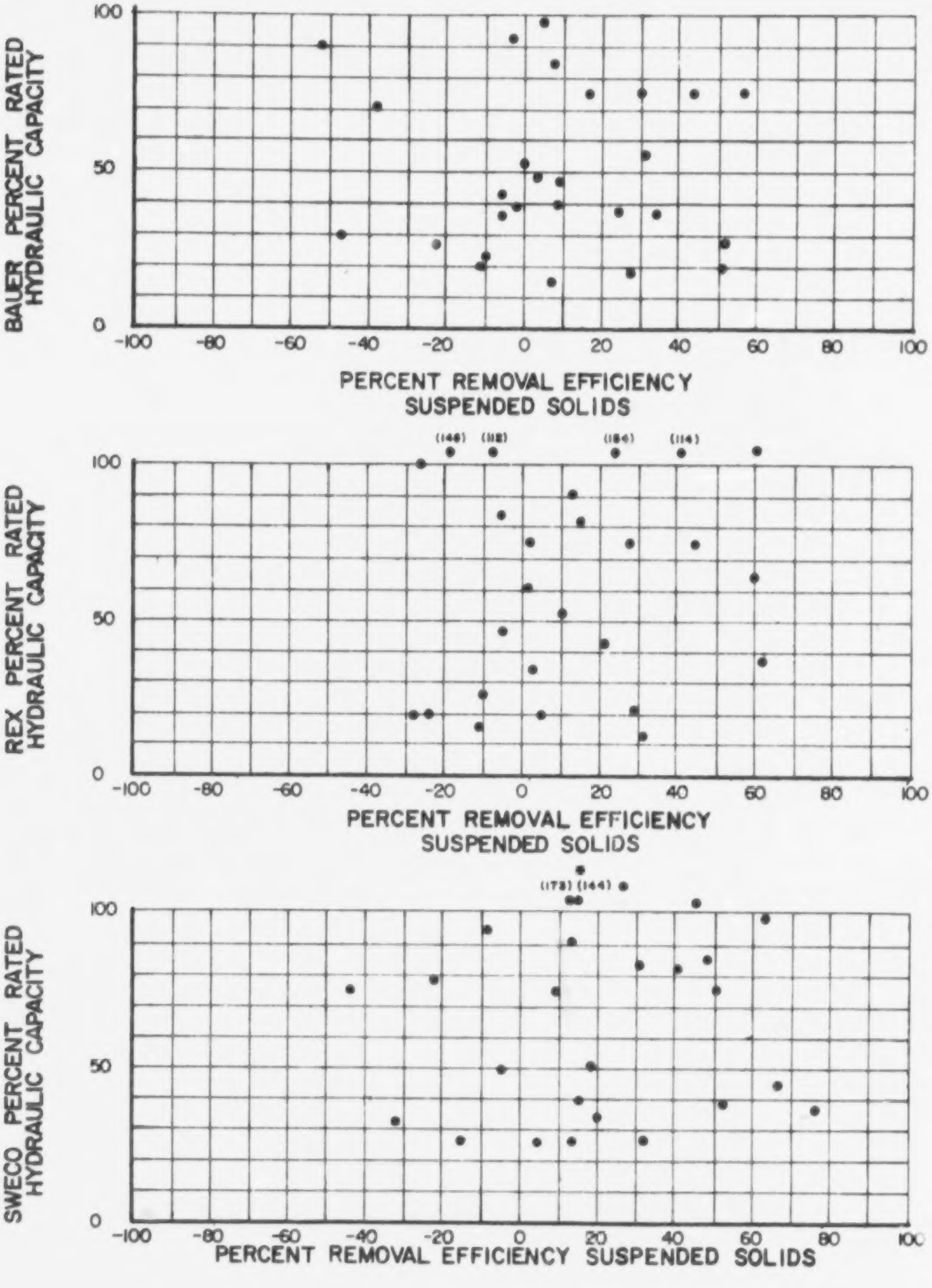


FIGURE 31 - SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR SUSPENDED SOLIDS (NON-FILTERABLE RESIDUAL AS A FUNCTION OF HYDRAULIC LOADING RATE)

As can be verified from the examination of this figure, there does not appear to be a discernable correlation between removal efficiency for suspended solids and hydraulic loading rate for any of the screens studied.

Another analysis parameter that was examined in this program was the rate of suspended solids loading. It was hypothesized that the screens could be expected to perform differentially to the rate of solids loading for the units. The "scattergram" results for this analysis are shown in Figure 32.

The average removal efficiency for the respective screens is given in Table 16.

Table 16. SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR SUSPENDED SOLIDS (NONFILTERABLE RESIDUE)

	Percent Removal of Suspended Solids		
	Bauer Hydrasieve®	Rex Rotary Drum Screen	SWECO CWC® Centrifugal Wastewater Concentrator
Arithmetic Mean @ 50%	+4.7	+9.2	+22.6
Standard Deviation	34.8	27.6	29.5
Number of Observations	27	26	28

One complicating factor to a systematic analysis of these suspended solids results is the frequency of screen failures of the SWECO, and to a lesser extent, the Rex units. There were insufficient screen failure data taken in this program to warrant the analysis of screen failure cases versus total cases for some discernable trend based on screen failures. In a strictly qualitative manner, there were several events observed where the fact of a screen failure was shown to have resulted in a decreased quantity of screenings concentrate for the SWECO CWC® units. This result is mentioned as a possible plausible reason for the wide dispersion of efficiency observed for this particular screen. However, refer to the later discussion pertaining to the screen life of these SWECO screens.

The available data for the removal efficiency of the screens for volatile suspended solids as a function of hydraulic loading rate show essentially the same random patterns as Figure 31. Similarly, the screening efficiency for the solids loading rate also follows closely Figure 32. Therefore, the volatile suspended solids efficiency data arrays are not shown. Table 17 presents the averages and estimates of dispersion for these volatile suspended solids data.

There were no data taken for any of the screening methods utilizing chemical flocculant or coagulant aids. However, data taken elsewhere indicate that these aids may improve screen performance. It was not possible to investigate these chemical aids under this program due to time and resource

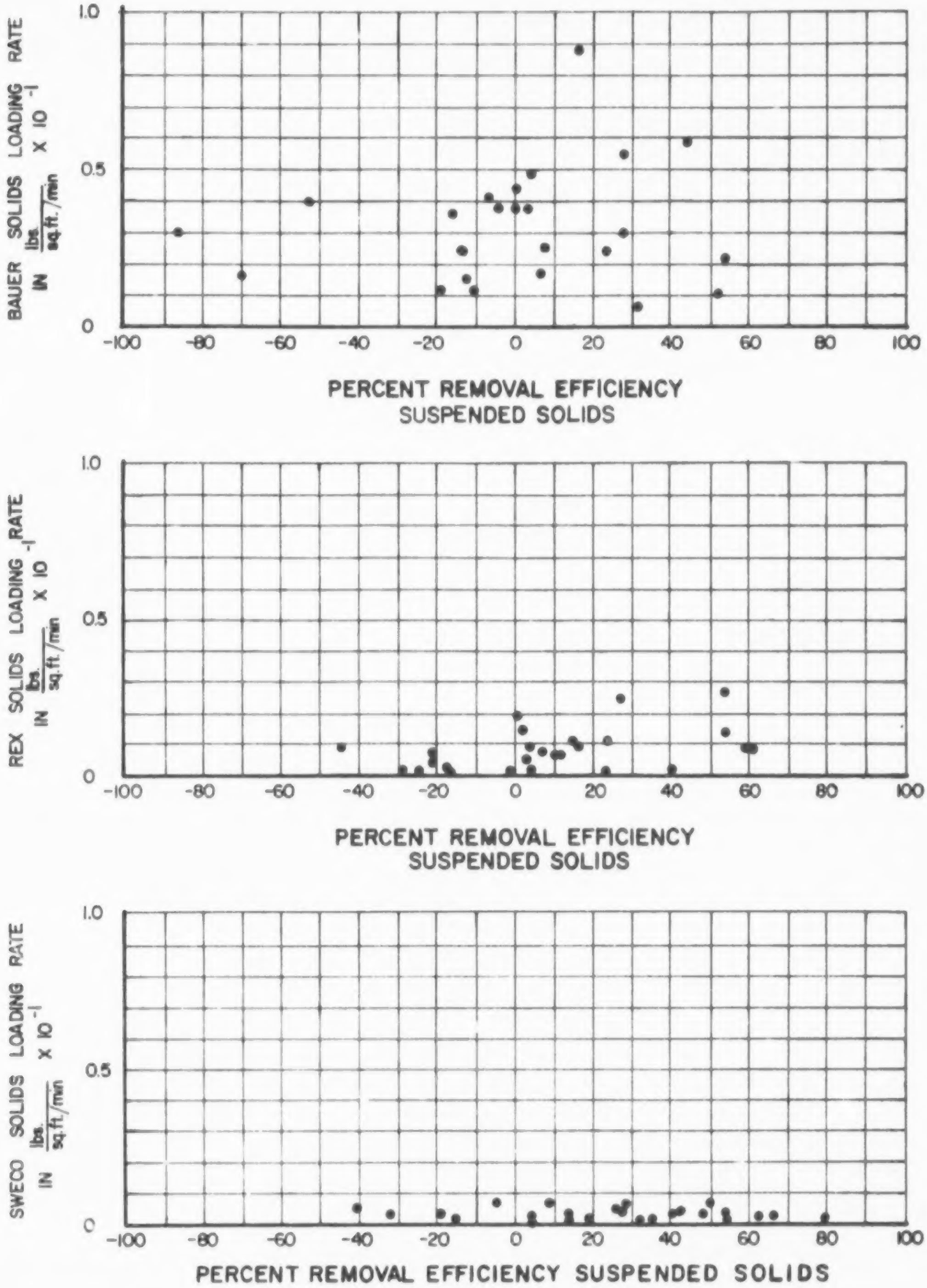


FIGURE 32 - SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR SUSPENDED SOLIDS (NONFILTERABLE RESIDUE) AS A FUNCTION OF SOLIDS LOADING RATE

limitation. There were no provisions for chemical additions at the Stormwater Facility except for chlorination and cleaner solution addition.

Table 17. SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR VOLATILE SUSPENDED SOLIDS (VSS)

	Percent Removal for Volatile Suspended Solids		
	Bauer Hydrasieve®	Rex Rotary Drum Screen	SWECO CWC® Centrifugal Wastewater Concentrator
Arithmetic Mean @ 50%	3.0	1.0	24.0
Standard Deviation	32	36	30
Number of Observations	23	23	25

Biochemical Oxygen Demand (BOD_5)--

The removal efficiency of BOD_5 was also examined. Shown in Figure 33 is the BOD_5 loading removal efficiency as a function of the hydraulic loading rate. The dispersion of the results shown in this figure is similar to that for the suspended solids data already examined in Figure 31.

Figure 34 is a figure depicting the BOD_5 removal efficiency as a function of BOD_5 loading rate. No apparent discernable trend can be identified from this analysis.

Table 18 presents the summary results for the BOD_5 loadings studied in this program.

Table 18. SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR BIOCHEMICAL OXYGEN DEMAND (BOD_5)

	Percent Removal of Biochemical Oxygen Demand		
	Bauer Hydrasieve®	Rex Rotary Drum Screen	SWECO CWC® Centrifugal Wastewater Concentrator
Arithmetic Mean @ +50%	+5.7	+3.4	+3.1
Standard Deviation	26.9	32.3	39.1
Number of Observations	26	25	27

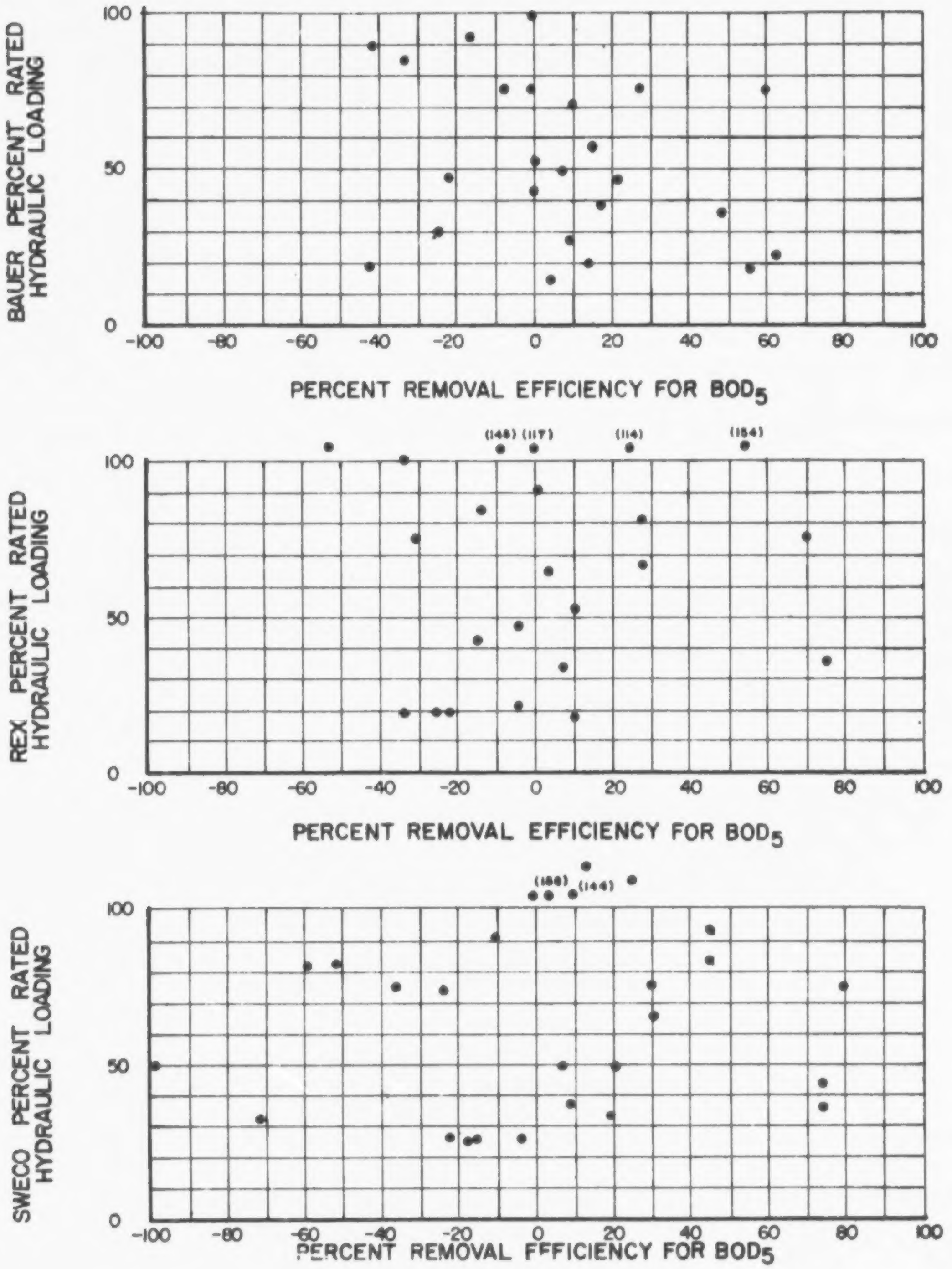


FIGURE 33 - SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR BIOCHEMICAL OXYGEN DEMAND (BOD_5) AS A FUNCTION OF HYDRAULIC LOADING RATE

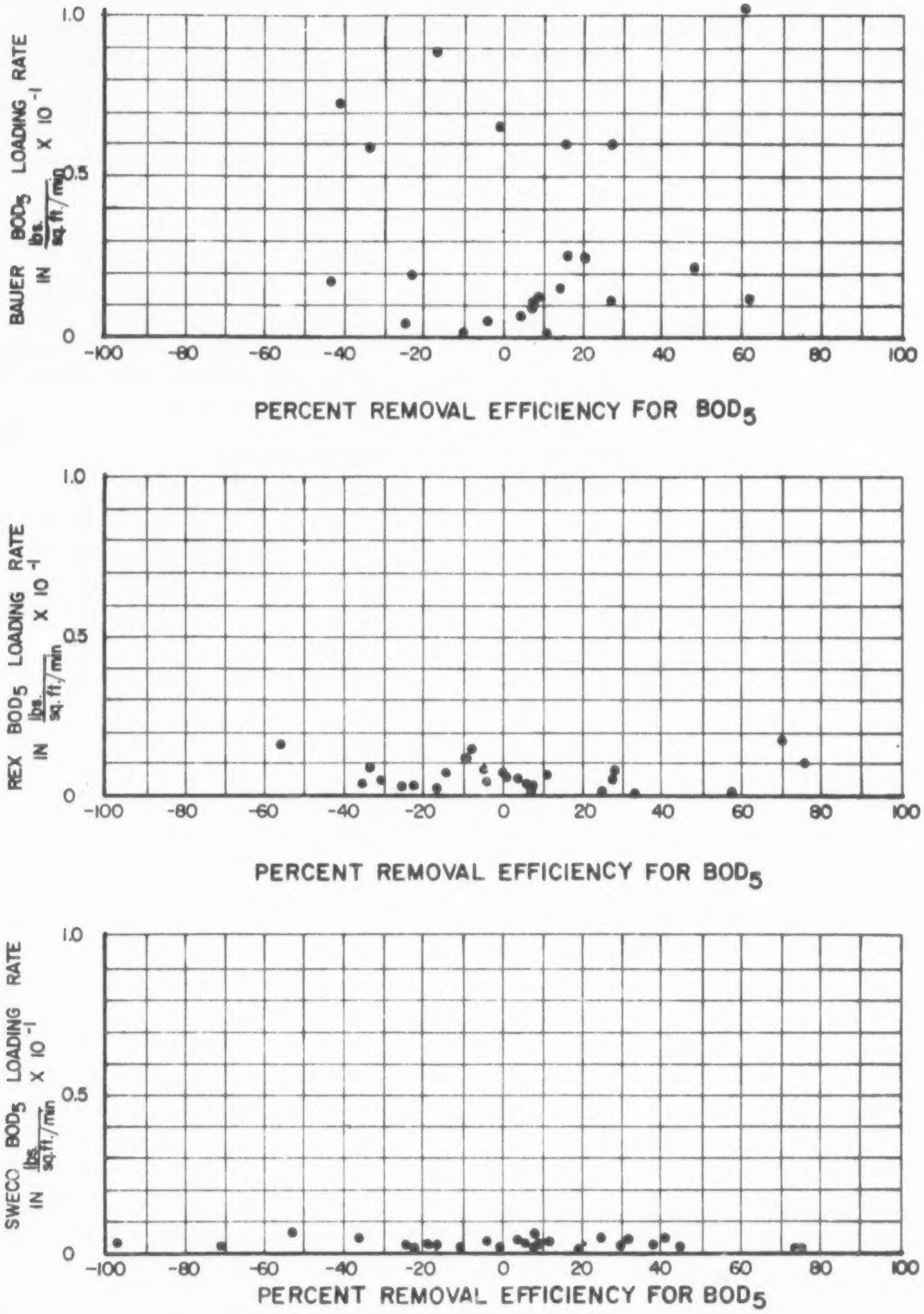


FIGURE 34 - SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR BIOCHEMICAL OXYGEN DEMAND (BOD_5) AS A FUNCTION OF BOD_5 LOADING RATE

These results are somewhat surprising because they imply that the Bauer Hydrasieve® with a screen opening size of 1.5 millimeters (0.060 inch) is on the same order of magnitude as the finest screen studied, SWECO CWC® with a screen opening size of 105 micrometers (0.004 inch). However, it should be noted that the difference between the mean efficiencies for the Bauer and the others is less than 2.0 percent, when the sampling and measurement error alone is not likely to be less than 25 to 35 percent.

Table 19 presents results for both total and filtered BOD_5. These data are based on event-composited samples taken every fifteen minutes over a 2-1/2-hour sampling period. Note that the filtered and total screened effluent BOD_5 in every case except the chlorinated screened effluent channel shows a gain in BOD_5 concentration as compared to the raw stormwater concentration. However, the magnitude of the gain in concentration is within the error of measurement for all samples except the Rex effluent total BOD_5.

Table 19. SCREENING FACILITY FILTERABLE BOD_5 DETERMINATIONS - EVENT NO. 207

	Event Composited Biochemical Oxygen Demand Total (mg/l)	Event Composited Biochemical Oxygen Demand Filtered (mg/l)
Raw Stormflow at Head Tank	109	82
Bauer Effluent	112	95
Rex Effluent	135	94
SWECO Effluent	119	85
Chlorinated Screened Effluent Channel	77	74
Rex Concentrated Screenings	195	--
SWECO Concentrated Screenings	112	84
Combined Facility Concentrated Screenings	145	--
Number of Aliquots	8	8

Chemical Oxygen Demand (COD)--

The "scattergram" for the removal of COD as a function of hydraulic loading rate is shown in Figure 35. The dispersion of results shown in this figure approximately equals that of Figure 33 for BOD_5.

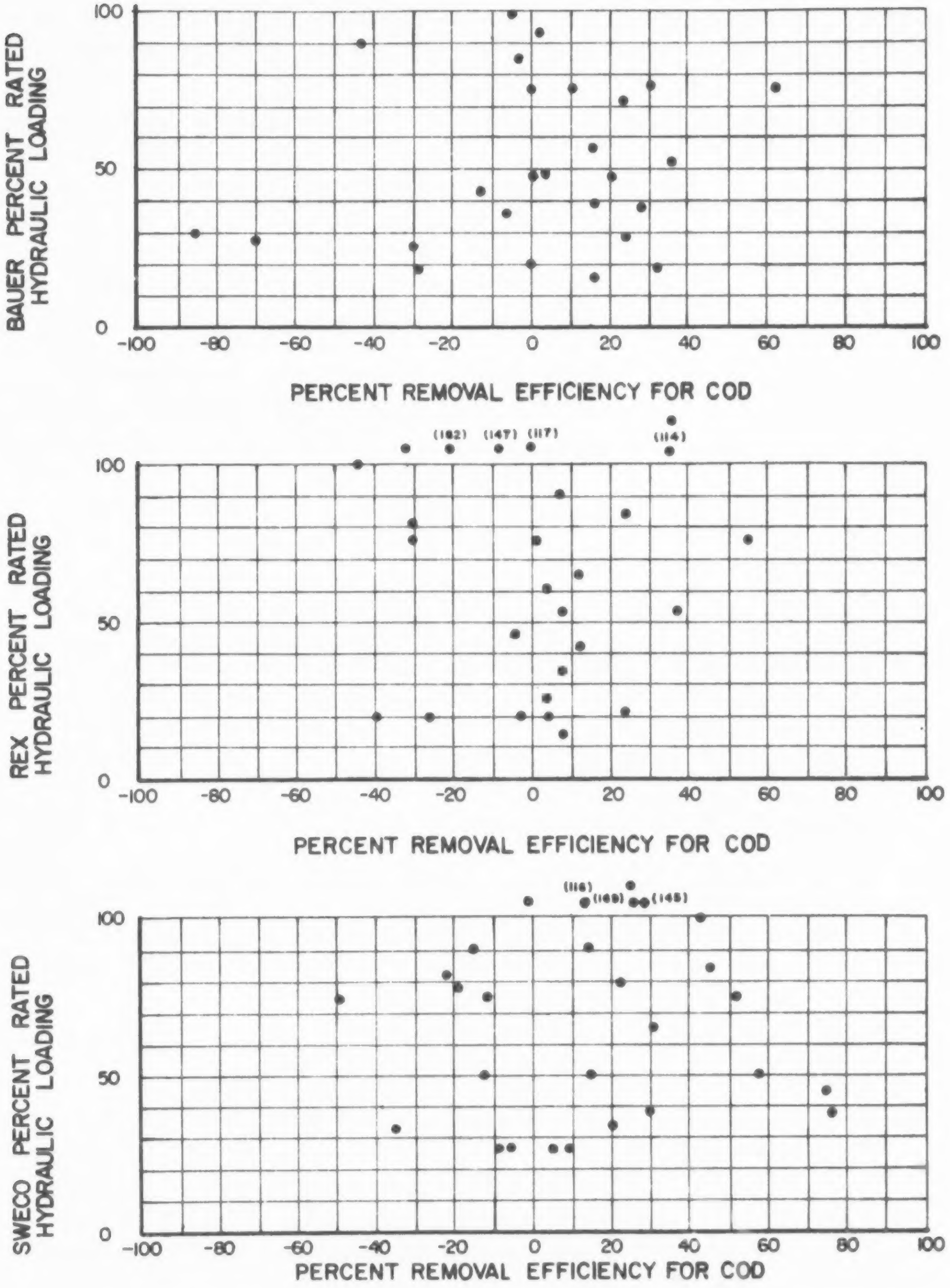


FIGURE 35 - SCREENING EQUIPMENT REMOVAL EFFICIENCY CHEMICAL OXYGEN DEMAND (COD) AS A FUNCTION OF HYDRAULIC LOADING RATE

Figure 36 is a "scattergram" showing the dispersion of COD loadings as a function of the unit COD loading rate. This result is similar to Figure 34 in that there does not appear to be any discernable trend between treatment efficiency and unit loading rate for COD as well as BOD_5.

Table 20 represents the statistical summary of these results for COD removal treatment efficiency.

Table 20. SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR CHEMICAL OXYGEN DEMAND

	Percent Removal of Chemical Oxygen Demand		
	Bauer Hydrasieve®	Rex Rotary Drum Screen	SWECO CWC® Centrifugal Wastewater Concentrator
Arithmetic Mean @ 50%	+6.5	+1.5	+12.9
Standard Deviation	25.6	27.3	29.2
Number of Observations	28	25	28

These statistical results run contrary to the trend of values for the BOD_5. They show the SWECO units have the potential for a higher removal efficiency on this particular waste parameter, which is definitely contrary to the result inferred from the data for BOD_5. This result may be due to the interaction of the raw stormwater wastes and the short-method COD test used in this testing program or some other undetermined cause. No data on the filtered COD versus total COD was taken in this program.

Total Phosphorus (Total P)--

Data taken for the screening equipment removal efficiency for total phosphorus are presented in Figures 37 and 38 as a function of hydraulic loading rate and total P loading rate, respectively. The dispersion of results for this parameter imply a differential removal from event to event and for hydraulic loading.

Table 21 is a tabular presentation of the statistics summarizing the removal efficiency data for total phosphorus.

The Bauer and Rex results for this parameter were sufficiently well-distributed to show a statistically significant result at the 95 percent confidence interval. However, the SWECO result did not pass the test for statistical significance at 95 percent confidence interval. Additional data is required for the SWECO corroborating the hypothesis these data in reality represent two separate subsets of data, one for the SWECO screens intact and another for the SWECO screens ruptured.

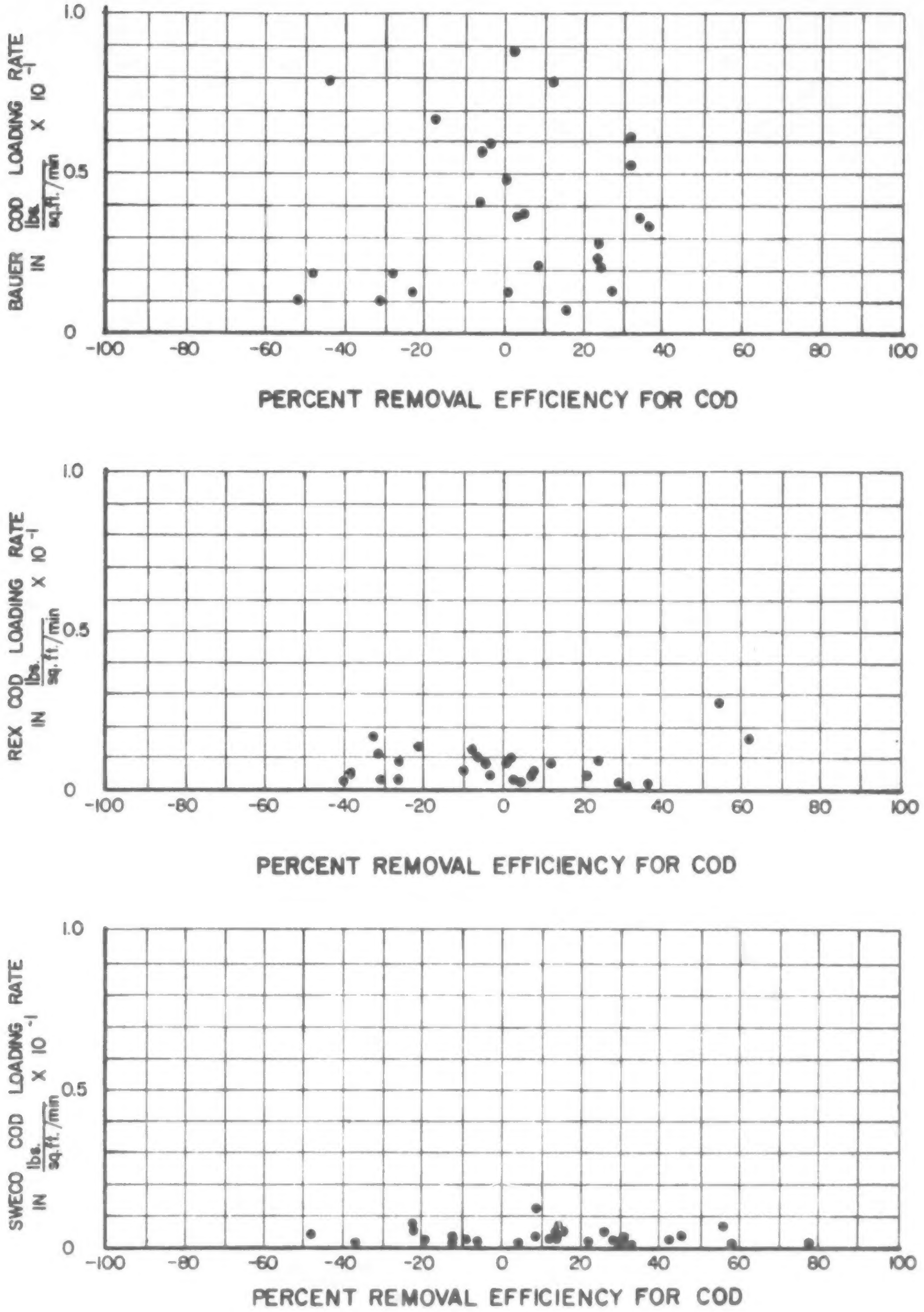


FIGURE 36 - SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR CHEMICAL OXYGEN DEMAND (COD) AS A FUNCTION OF COD LOADING RATE

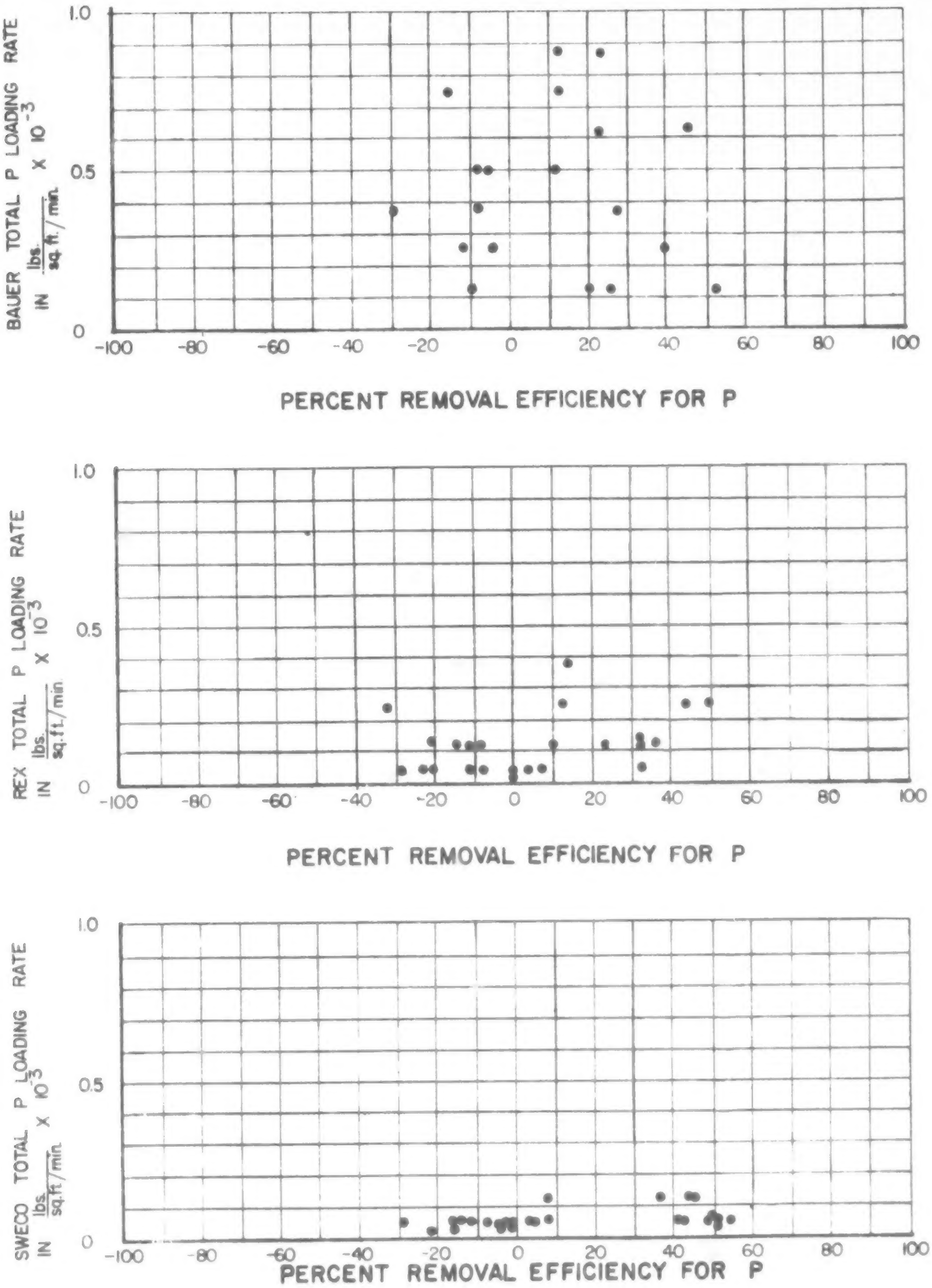


FIGURE 37 - SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR TOTAL PHOSPHORUS (TOTAL P) AS A FUNCTION OF HYDRAULIC LOADING RATE

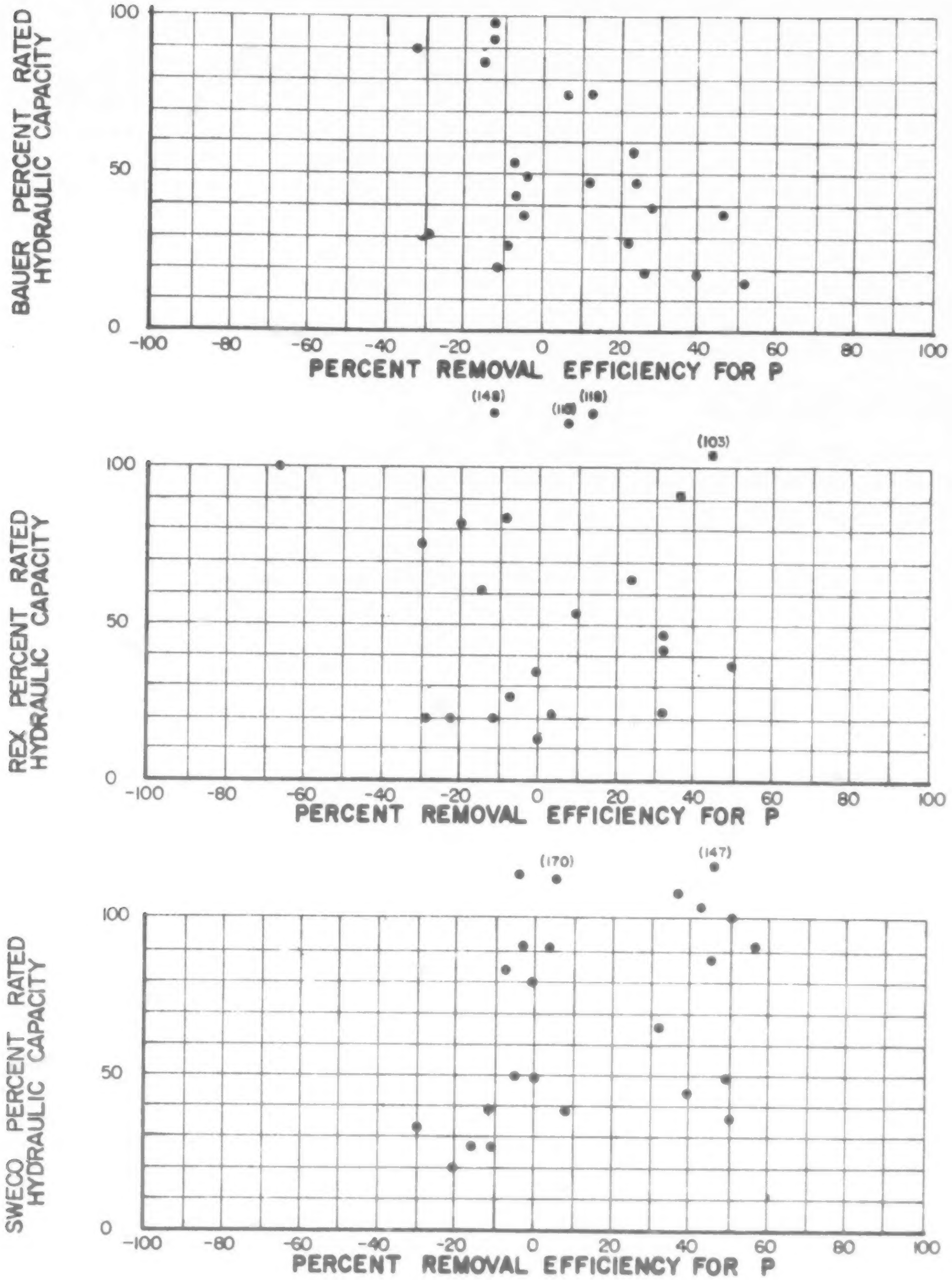


FIGURE 38 - SCREENING EQUIPMENT EFFICIENCY FOR TOTAL PHOSPHORUS (TOTAL P) AS A FUNCTION OF TOTAL P LOADING RATE

Table 21. SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR TOTAL PHOSPHORUS (Total P)

	Percent Removal of Total Phosphorus		
	Bauer Hydrasieve®	Rex Rotary Drum Screen	SWECO CWC® Centrifugal Wastewater Concentrator
Arithmetic Mean @ +50%	+6.9	+2.0	+8.3
Standard Deviation	23.5	27.8	47.5
Number of Observations	24	23	25

Ammonia Nitrogen (NH_4^+-N)--

Data taken for ammonia nitrogen were not sufficient to warrant analysis by dispersion analysis using a "scattergram". A mean value was determined for the three screening methods studied and the appropriate statistical summary parameters are given in Table 22.

Table 22. SCREENING EQUIPMENT REMOVAL EFFICIENCY FOR AMMONIA NITROGEN

	Percent Removal of Ammonia Nitrogen		
	Bauer Hydrasieve®	Rex Rotary Drum Screen	SWECO CWC® Centrifugal Wastewater Concentrator
Arithmetic Mean @ 50%	-4.9	-3.5	+15.3
Standard Deviation	25.9	20.4	26.4
Number of Cases	17	17	17

Partly due to the few number of cases considered, ammonia nitrogen removal efficiency data are difficult to interpret. The result for the Bauer and Rex units is statistically significant at the 95 percent confidence interval, whereas the result for the SWECO does not come out as statistically significant at this confidence level. Examination of the frequency of occurrence analysis for the SWECO tends to indicate a bimodal distribution with the modal category at +5 for one subset and +25 for the other. Whether this result is an inherent anomaly of the small number of cases or conversely an indication of the screen failure mode is unclear because these data are insufficient to determine. However, it should be noted that the removal of ammonia nitrogen by screening is not likely to be plausible grounds for supporting the screen failure hypothesis because ammonia nitrogen is inherently a soluble pollutant. This tends to discredit any inference that there is some removal efficiency attributable to the physical process of screening.

Fecal Coliform Group Indicator Bacteria--

This parameter was monitored briefly as a potential screening removal parameter but the results of an eight-week trial on this parameter did not indicate any measurable removal of fecal coliform as measured by the membrane filter method. Thereafter, the data-gathering for bacteriological sampling was terminated as unproductive. Fecal coliform monitoring of other sampling points in the treatment scheme continued for the duration of the program.

Screening Efficiency Net Overall Treatment Efficiency--

In summary, considering only the data resulting in a positive treatment efficiency, Table 23 represents the net overall treatment efficiency summary for the screening equipment evaluated.

Table 23. SCREENING EQUIPMENT NET OVERALL TREATMENT EFFICIENCY IN PERCENT (± 95% C.L.)

	Bauer Hydrasieve®	Rex Rotary Drum Screen	SWECO® CWC
Suspended Solids	15.7 ± 19.0	15.8 ± 19.9	26.2 ± 23.2
Biochemical Oxygen Demand	13.1 ± 18.4	14.4 ± 23.1	16.5 ± 21.9
Chemical Oxygen Demand	13.6 ± 16.2	11.3 ± 17.8	19.5 ± 20.7
Total Phosphorus	13.3 ± 16.4	11.4 ± 15.5	18.9 ± 21.0

Operations, Maintenance and Special Considerations

From the original proposed Plan of Study for this work, the evaluation of the functional value of the equipment installed was a principal objective of this program. The Statement of the Work[4] for the program specifically described the scope of the study as:

> 6. "Determine the efficiency and effectiveness of the individual and collective overflow treatment facilities with respect to pollution abatement, cost effectiveness, reliability and other aspects of evaluation..."

This section is intended to present those observations and results that fall into the general category of "...effectiveness, reliability and other aspects of evaluation...". These results specifically pertain to the Fort Wayne Stormwater Facility and may be unique to this particular facility.

Stormwater Facility Conduits and Wetwell--

The stormwater conduits constructed under this grant have been in continued service throughout the duration of this program. Operations have been at lower transit velocity than the design hydraulic capacity of these sewers, designed for 1990 service at five times the present day hydraulic loading. This set of circumstances has contributed to the sedimentation of suspended solids throughout the conduits and the wetwell.

Compounding this circumstance of future design loadings is the fact that the wetwell has been partly full of wastewater since start-up in January, 1975. The wetwell dewatering submersible sump pump was found to be unable to dewater the wetwell and the conduits. Several events where the turbulence of flow in the conduit churned up heavy septic, malodorous sediments were experienced in the evaluation period. These sediments were sufficiently concentrated to blind the mesh of the drum screen and the SWECO units. This usually backed up in the flow distribution manifold to the head tank.

The water surface would increase in the head tank or the Rex headbox until it exceeded the overflow discharge elevation, which then carried these raw, untreated flows directly into the screened effluent channel. These sediments are thought to account for the sludge banks in the chlorine contact channel shown in Figures 29 and 30, preceding.

Stormwater Pumps--

There were no exceptional operational difficulties experienced with either the Johnston Model 24 PS (1.10 cumec or 25 MGD) or 30 PS (2.19 cumec or 50 MGD) stormwater pumps. From calibrated flow measurement points installed in the facility, these units performed adequately on a hydraulic basis throughout the evaluation period.

Stormwater Distribution System--

This subsystem presents few operational problems except to note that the overflow in the head tank passes untreated to the effluent channel carrying the screened wastewaters. The manually operated knife gate valves used in the flow routing conduits as the flow throttling devices required little maintenance and were sufficiently easily operated to adequately control the equipment flows.

Bauer Hydrasieve®--

The Bauer unit is inherently simple to operate because it has no moving parts. However, the unit supplied at Fort Wayne experiences some hydraulic limitations due to influent channel turbulence. A sudden enlargement hydraulic jump phenomenon caused excessive loss of contaminated raw stormwater to the floor drain system by presenting a falling water curtain under the unit. This hydraulic distribution problem was field-corrected by the manufacturer after several months of operation.

The Bauer unit required so little in the way of routine maintenance after each event that it gradually built up a layer of oily or greasy material on the screen. This layer was gummy wet or dry and required cleaning by detergent application and manual scrubbing from time to time.

Adjustment of the flow distribution baffles at the top of the screen was possible for increasing the flow handling capacity. However, the unit hydraulic loading was typically varied by the pumping condition and the desired flow split from the head tank to all the screening units in service at the time. Some type of flow control to each module based on an optimum hydraulic flux rate per screen module would probably have improved the performance of this unit in the end.

Some provision for the removal of semidry screenings directly for ultimate disposal such as a solids collection hopper dumping into a containerized disposal vessel would have been very desirable. These screenings are already washed (like clean grit) and frequently could be raked off the Bauer screen surface. Instead, operators were required to hose down the unit with plant water and this created a lot of unnecessary splashing and mess.

The Bauer unit makes an excellent pretreatment device for the finer screens such as the SWECO and the Rex. It removes solid debris such as wood pieces, metals, plastic materials, leaves and twigs, and virtually any other coarse solids. These materials were shown to cause one screen failure after another for the SWECO.

The Bauer screen modules were exclusively operated in parallel with each other. Over the evaluation period, these units were responsible for treating 113.5 million gallons (430 million liters). The Bauer was always ready to go into service and served as a type of "surge device" for distribution of flows to the other units. It was operating a total of 276.1 hours for an average duration of 3.2 hours during a total of 86 events.

No data were taken on the amount of screened effluent aeration that could be attributed to the Bauer unit. However, like each of the other screens, the effluent from this unit showed quantities of frothy suds coming out of the effluent trough.

On the basis of floor area required, this unit takes up a space of 8.45 meter by 7.85 meter (27'-9" by 25'-9"), or 66.3 square meters (714.6 square feet). Since it is elevated above the grade level of the screening room, the floor area beneath this unit is clear except for the support columns for the unit. There are no utility services required for this unit except for cleanup after an event. Headloss considerations are important for this method because the nature of the design requires not less than about 2 meters (6.6 feet) of static headloss to operate.

Rex Rotary Drum Screen--

The Rex Rotary Drum Screen was operational in 50 events in the post-construction evaluation period (January, 1975 to February 6, 1976). The unit operated a total of 176.1 hours for an average running period of 3.0 hours The unit was inoperative for the first three months due to a bearing support frame problem that was field-corrected in April, 1975. Thereafter, throughout the duration of the evaluation period, this equipment was plagued with mechanical problems.

One of the most vexing problems with this specific unit was its tendency to cease rotation under hydraulic loading. This problem occurred so frequently and under such varied operational conditions that it was extremely difficult to identify the precise reason for the problem. Sometimes the drum drive would start to stall because the V-belt drive sheave was partly submerged. However, at other times, under similar operating conditions, there were no slippage problems.

Another source of slippage of the drum drive was identified as the increased friction from the drum support roller wheels located at the open headbox end and shown in Figure 12, preceding. These wheels are essential idler wheels which transmit the drum weight to the support beam shown in the photograph. A grease-lubricated roller-type bearing is provided in the hub of the wheel. The roller bearing in one unit seized under load sometime after the tenth month of operation and increased the rolling friction of the drum. This resulted in wear to the roller surface, necessitating replacement, and was also responsible for some slippage of the drive sheave.

At times, the drum would slow and slip without completely stalling. These occasions were usually observed when high differential head between influent and effluent flows occurred. This condition was also observed to be the result of a dirty drum screen. An overflow weir, shown previously in Figure 28, was provided to relieve the hydraulic head that would result. Normal operational practice under such conditions was to throttle back the drum influent flow to allow the automatic backwash system to clean the drum.

Problems with the backwash system itself were also experienced. The system as installed used the screened effluent as a make-up source for the backwash pump. However, the overflow relieves to the same channel as the effluent. This condition resulted in plugging the pump strainer with debris at times or, more frequently, plugging of the backwash manifold nozzles which had to be manually cleaned. This problem was solved by putting the backwash pump make-up onto the plant water system which was connected to the potable water system through a break tank.

The Rex unit experienced one screen panel failure in the evaluation period resulting from a puncture or tear from a small wood block of scrap that had become trapped in the drum. This unit required routine maintenance after an event to clean up, inspect the backwash system, drive mechanism, etc.

The Rex drum screen also experienced two separate main bearing failures. The main bearing for this unit serves as the main support for the drum on the closed end as well as the thrust bearing to resist the hydraulic flow-induced axial forces against the closed end. Both failures were attributed to excessive deflection by the bearing support. This problem was corrected in the field by the manufacturer.

On the plus side, the Rex drum screen takes up less floor space than either of the other two screens. The power requirements to operate this unit are modest by comparison with the SWECO. Perhaps the most significant advantage to the rotary drum screen is the much lesser quantity of concentrated screenings to be handled.

Figure 39 shows a photograph of the Rex Rotary Drum Screen operating on recirculated pond water. The screened effluent overflow point shown in this photograph also represents the flow measurement weir for this unit, as calibrated in the post-construction evaluation flow measurement program.

The Rex unit at the Fort Wayne Stormwater Facility sits in a poured-in-place concrete basin which takes up a floor area equivalent to 6.09 meters by 6.25 meters (20'-0" by 20'-6"), or 38.05 square meters (410 square feet). It requires an electrical power supply for the drive motor and the backwash pump motor plus controls, a clean source of backwash make-up water, amounting to 378 liters per minute (100 gallons per minute) at a discharge of 45.8 meters (150 feet) TDH. Manual cleaning of the screen wire fabric with scrub brush and chemical cleaner may be necessary periodically to clean up the accumulated greasy film that eventually builds up on the screen.

Figure 39 - Rex Rotary Drum Screen Operating on Recirculated Pond Water

The Rex unit can be operated at the least static lift headloss among the screens evaluated. A minimum headloss of about 0.75 meters (2.4 feet) is required for the unit at Fort Wayne.

Data presented elsewhere indicate a significant improvement in overall treatment may be expected when a chemical flocculant was used in screening/flotation processes. The chemical flocculant addition necessary to achieve the removal efficiency improvements cited was 20 milligrams per liter ferric chloride and 4 milligrams per liter cationic polyelectrolyte.[5]

These amounts would have required considerable quantities of these chemicals to be added to the flows treated at Fort Wayne. For example, to add 20 milligrams per liter of ferric chloride to a flow of 17,500 gallons per minute (66,200 liters per minute) would require the addition of 2.9 pounds per minute (1.3 kg/minute), or the equivalent of one 135-pound (6.12 kilogram) drum of anhydrous ferric chloride every 45 minutes. Since this material is very hazardous to handle in either bulk liquid or anhydrous form, no provisions were made to add chemical coagulants to the flows at Fort Wayne.

The Fort Wayne Facility combined sewer overflows are located at the Water Pollution Control Plant (the 84-inch bypass) and the Glasgow Street interceptor regulator. Both sources are extensively used to store plant influent flows under many operating conditions. The use of these conduits as storage volume results in both flow and concentration damping of pollution surges represented by storm-caused first flush conditions. In addition, there are major industrial contributors on these interceptors, particularly the Glasgow Street conduit. As a practical matter, some or all of the plant raw sewage flow can be sent to the Stormwater Facility. The absence of the storm-caused first flush overflow condition cited elsewhere is a direct resultant of these factors. Consequently, addition of flocculant aids to these wastes probably would not materially improve the performance of the screens, including the Rex, based on past plant experience in treatment of primary influent with coagulant aids.

SWECO CWC® Centrifugal Wastewater Concentrator--

The SWECO units operate on the basis of rotary fine mesh screening of a sheet of raw wastewater hitting the collar screens. Particles suspended in the flow are entrapped on the mesh of the screen while the screened fluid is allowed to pass on through the screen and drops into a screened effluent channel. The separated particles are washed down to produce the concentrated screenings which are collected in a separate channel for conveyance to the Water Pollution Control Plant for additional treatment.

The units installed at Fort Wayne were intended to be capable of automatic operation with manual override feature. From the beginning of the evaluation period, it was impossible to operate these units in the automatic mode without causing problems with the flow distribution system because frequently all eight CWC screening units began their backwash cycles simultaneously. This reduced the flow treatment capability of the SWECO screens essentially to zero for the duration of the backwash cycle.

The automatic operation feature for these screens originally was triggered individually by the level of concentrate in the screenings concentrate channel. Sometimes, the first flow of each unit experienced was so heavily laden with heavy, greasy material that the units one by one would increase the quantity of concentrate to such a level as to allow very little of the raw stormwater to pass through the screens into the screened effluent channel. With most of the influent flow from one or more screens going to the concentrate channel, the level in the channel would rise sometimes to overflowing and all units would sense the high level in the backed-up channel until the sensor called for the influent valve to close automatically.

Another problem created by this simultaneous backwash demand of more than two or three units at a time was the high peak short-term demand for compressed air in the influent valve pneumatic operator circuits. The compressor reservoir would be exhausted and the pump could not catch up on its own. This problem was corrected by adding additional reservoir capacity for the pneumatic air supply system.

The manufacturer's solution to the continuous backwash demand problem was to rewire the electrical control panel to put a master cycle timer in the individual backwash timer such that when a unit on one bank of CWC screens called for backwash by timer, all other units were essentially "locked out" for calling for backwash until the backwash timer cycle on the unit being serviced timed out. This arrangement limited the number of units in the automatic mode calling for backwash at a time to one for each bank of CWC units. This arrangement worked satisfactorily for the remainder of the duration of the evaluation period as long as the units started with clean screens.

It became necessary at each event shutdown to ensure there was a clean CWC ready for service at the next event start-up. To accomplish this, the unit was operated in continuous cold water backwash all the time. This improved the length of runs between backwash requiring hot water somewhat. The operating personnel were instructed to manually initiate a backwash cycle for one unit at a time when it became obvious that the screened effluent had diminished and the concentrate had increased. At the termination of each event, each CWC, whether operated or not, was manually put through a hot water backwash cycle three complete times as a part of clean-up standard operating procedure.

Frequently, the CWC units were operated with manual backwash cycles every thirty minutes or so.

Another operational difficulty experienced with the SWECO screens was the amount of concentrated screenings generated by the units. This problem was aggravated by the fact that the Stormwater Facility concentrate drains were located on the Lakeside interceptor, a combined sewer. This sewer was subjected to stormwater surcharge just when the necessity of removal of concentrated screenings was greatest. The result was in effect to increase the amount of head needed to "push" the screenings into the sewer or limit the number of SWECO's on stream at a time. When this head requirement became greatest, the concentrated screenings backed up the channels in the screening room and overflowed the channels onto the floor. This result effectively reduced the weir flow measurements of concentrated screenings to nothing, and the screening equipment evaluation required an assumed flow split of 15 percent to concentrate, 85 percent to the effluent channel.

The most significant feature of operations and maintenance of the SWECO screens stems from the screen failures experienced during the evaluation period. There were 142 screen failures identified in the evaluation period, in which the combined hours of operation of these units totaled 1,500 hours. A failure of a single screen in a single CWC unit usually resulted in a drastic increase in the effluent pollutant loadings for the remainder of the screening event.

The computation of screen life is based on the principles of reliability engineering. Reliability is defined as "the probability that a system or device will perform without failure under given conditions for a specified period of time". A computation of the Mean Time Between Failure (MTBF) was performed for any one CWC unit, and the result is carried through the calculation of the expected useful life per screen; $t = 30.5$ hours.

This result is a direct consequence of the nature of the wastes and the manner of distribution of the raw wastewater. For the first twelve months of the evaluation period, the raw stormwater flow was distributed to the SWECO screens directly from the head tank. This period represents the majority of the screen life failures simply from the number of hours of operation. The remaining six weeks in the evaluation period was the only period where the Bauer Hydrasieve®was used as a prescreening device for the north bank of SWECO screens. Screen life results for this period were not sufficient to extrapolate a different useful life for this tandem operational mode. It is expected that the failure mode for the north bank of units will shift to a decidedly more normal curve distribution that approximates a classical wear out failure rather than a chance failure mode. This result is expected to extend the useful life of each screen to several hundred hours or more. However, it should be apparent that the south bank of units will probably continue in high chance failure mode because there is no provision for prescreening any of these units' influent flow.

The first few months of operation (without the continuous backwash spray in service) resulted in heavily blinded screen panels from greasy deposits thereon. These deposits adversely affected the hydraulic through-put of the screens. It was necessary to manually scrub each screen panel (36 for each CWC unit) with a diluted muriatic acid solution to restore the clean screen porosity. This was recommended by the manufacturer prior to the installation of the continuous backwash arrangement. After completion of this manual cleaning and the continuous backwash system, manual scrubbing of these panels was not required.

Figure 40 is a photograph of the SWECO effluent showing the sudsing action of the screened stormwater. This sudsing came and went within events, and some events did not show any sudsing action. As a general rule, once the sudsy effluent left the Screening Building, whether chlorinated or not, the suds disappeared when exposed to wind and weather.

The eight SWECO screens installed at the Fort Wayne Stormwater Facility sit overhanging the screened effluent channel. Counting the channel area required to carry the flows out of the Screening Building, these units as installed require an area of 5.5 meters by 17.8 meters (18'-0" by 58'-6"), or 97.7 square meters (1,052 square feet). This area can be oriented in several ways. However, the overhead requirements of a SWECO installation require a distribution channel capable of delivering the raw inflow at a recommended static head of 2.5 to 3.0 meters (8 to 10 feet). Figure 41 shows one of the two distribution manifolds for the SWECO units. This manifold is a pressure conduit operating at 1.8 to 3.7 meter static head (6 to 12 feet) and delivering the flow to each of the units along its length in turn.

The SWECO units require more auxiliary services for operation than the other screens. Each unit has a pneumatically operated 12-inch gate valve with electrical solenoid valve actuators. Each unit also has a chemical feed pump and a drive motor. An air compressor with receiver is required for the pneumatic valves' air supply. Some source of clean make-up water for backwashing is required. At Fort Wayne, this was originally to be pond water but the pumping requirements and solids plugging problems required the use of potable water for backwash water make-up. A hot water heater for the screens' hot water backwash is required. Chemical cleaning solution concentrate is necessary.

The necessity of all these auxiliary services for the SWECO required that the Fort Wayne Stormwater Facility be kept heated during the cold weather months to about 10 degrees Celsius (50 degrees Fahrenheit). At one time in January, 1975, the solenoid valves on several SWECO screens froze from moisture in the air line and ruined the actuators. Heating the building prevented future occurrences of this.

Stormwater Facility Chlorination--

The Stormwater Facility was designed with the capability of chlorinating at several points in the treatment scheme. As shown in Figure 5, preceding, it was possible to prechlorinate combined sewer overflows diverted to the Stormwater Facility by opening the valve at the 84-inch bypass structure or alternatively, the chlorination line drain valve at the facility wetwell. The practice of prechlorinating the stormwater overflows was discontinued after several operational personnel assigned to the Stormwater Facility complained of "tear gas" - like fumes in the screening room when either of the prechlorination points were in service.

The post-chlorination point was operated for 18 events to assess the impact of storm flow chlorination on the terminal pond. There are not sufficient data to substantiate the effect of chlorination on the pond one way or another. However, the pond effluent data taken since pond start-up in October, 1971, tends to confirm that there is no adverse effect of chlorination of pond influent.

Figure 40 - SWECO CWC® Screens in Operation on Raw Stormwater Overflows Showing Effluent Sudsing Effect

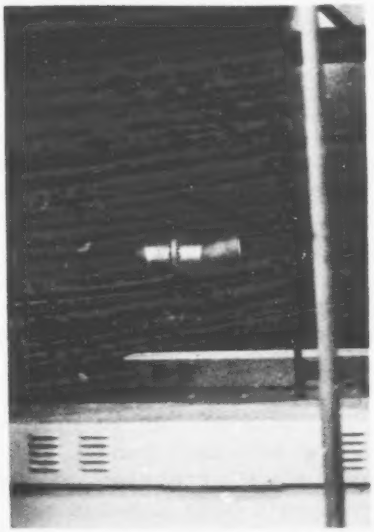


Figure 41 - SWECO Influent Distribution Manifold for South Bank of CWC Units

Chlorine demand for the post-chlorination point of application was highly variable, depending on the wastewater. A minimum chlorination rate of 6.0 kilograms per million liters (50 pounds per million gallons) was observed during an event in June, 1975. The maximum rate of chlorine application was 21.1 kilograms per million liters (176 pounds per million gallons). The mean rate was 13.3 kilograms per million liters (111.22 pounds per million gallons), standard deviation of 6.79 kilograms per million liters (56.69 pounds per million gallons).

Effectiveness of the Stormwater Facility in Dealing with Combined Sewer Overflows--

In the pre-construction program, it was calculated that 1,040 million gallons (3,935 thousand cubic meters) bypassed any sort of treatment by overflowing the 84-inch bypass sewer in at least 58 separate storm-induced combined sewer overflow events. This condition represents the baseline for comparison of the effect of the Stormwater Facility on the combined sewer overflow bypass diversions to the Maumee River. In the post-construction evaluation period, there were eleven events identified where some raw, untreated storm flow was relieved hydraulically by overflowing the Stormwater Facility wetwell. A total amount of bypassed stormwater overflow was estimated at 35.1 million gallons (133 thousand cubic meters) for this period.

The reasons for these bypasses varied. There were seven events where the duration of the bypass could have exceeded 60 minutes. The longest observed bypass came during Event 192 on September 5, 1975, when the two stormwater pumps simply could not keep up with the wetwell inflow resulting from a 1.08-inch (2.74 centimeter) rainfall. There were six other times where the bypass could have lasted as long as 4 hours. Because time required to man the Stormwater Facility and start up the raw stormwater pumps averaged less than one hour, the average duration of bypass was limited to 1.6 hours when there were problems with getting the facility going. There were four events where the time of bypassing was 30 minutes or less.

On the basis of the decrease in the frequency of diversion of storm-caused combined sewer overflows at the Water Pollution Control Plant alone, the construction of the Stormwater Facility was highly successful in controlling overflows. Furthermore, it should be recognized that the 84-inch bypass depth indicator at the WPCP recorded 117 separate diversions during the post-construction evaluation period. It is concluded that the construction program reduced the frequency of overflows to one-tenth of the number of potential overflows.

On the basis of the estimated quantity of raw combined sewer overflows diverted to the river, the Stormwater Facility was shown to have reduced the potential overflow of 450.9 million gallons (1,707 thousand cubic meters) to an estimated 35.1 million gallons (133 thousand cubic meters), an 82 percent reduction. In the original design concept, outlined in the Master Plan for Sewers, Part I,[3] it was estimated that the diversion of only 50 percent of the combined sewer overflow quantities received at the plant would result in a 10 percent reduction of the organic loading going to the stream. In this program, the reduction in frequency and quantity of flow bypassed amounted to a net estimated reduction of organic loading of 10,300 pounds of BOD_5 (4,700 kg of BOD_5) to the stream, a percentage reduction of 54 percent from this source alone.

Stormwater Effects on Terminal Pond

One of the objectives of this program was to assess the overall effect of storm-caused combined sewer overflows on the Water Pollution Control Plant terminal pond effluent. Figure 42 is a flow schematic diagram for the treatment plant systems showing the origins of flows from the various sources. The pond receives daily flows from the Water Pollution Control Plant on the south side of the river. These flows, designated Q_{WPCP}, are made up of the treated secondary effluent and the remaining primary effluent that is untreated by the aeration tanks due to hydraulic flow limitations. These flows are combined and chlorinated in the plant effluent diversion chamber at the effluent chamber of the secondary settling tanks on the south side of the river. By means of a 72-inch river siphon, these flows are conveyed to the influent of the chlorine contact tank at the east end of Terminal Pond No. 1.

The effluent from the chlorine contact tank flows through a 72-inch conduit to the pond headworks. A drop inlet for the Stormwater Facility tees into this pipe at the end of the screened stormwater effluent channel just before this conduit enters the terminal pond. This Stormwater Treatment Facility effluent flow, already screened and chlorinated, is designated as Q_{stm} on the flow diagram. As can be seen, in the flow diagram, the terminal pond influent is made up of the screening facility effluent plus the Water Pollution Control Plant effluent (Q_{WPCP}).

Once the combined Stormwater Facility effluent plus WPCP effluent entered the terminal pond, the plant operating practice was to monitor the pond effluent by 24-hour composite sampling at the WPCP for most of the pollutant parameters. Bacteriological samples, dissolved oxygen samples and temperature were taken once a day by plant operators on a grab sample taken in the mid-morning. These routine plant operating data were used in the evaluation of the effect of combined sewer overflow screening facility effluent on the pond performance. An analysis of terminal pond flow and detention time was an integral part of this analysis.

The starting point for the sizing of the Stormwater Facility was in the Master Plan for Sewers, Part I[3], which discussed the necessity of stormwater-caused combined sewer overflows, the benefits to be derived from construction of relief sewers and centralized treatment facility, and the means for treatment. The goals of the facilities to be constructed for the treatment of combined sewer overflows were to reduce the overall stream BOD_5 loading by 10 percent. This goal was to be accomplished by diverting the major part of the combined sewer overflow raw untreated wastewater to a stormwater treatment facility. The flow that was anticipated for treatment in such a facility was estimated at 600,000 gpm ultimately. The initial construction phase (undertaken under this project) was sized to handle 52,500 gpm or 75 MGD (3.29 cubic meters per second). This quantity of flow was to be obtained from the 84-inch bypass sewer overflow at the Water Pollution Control Plant.

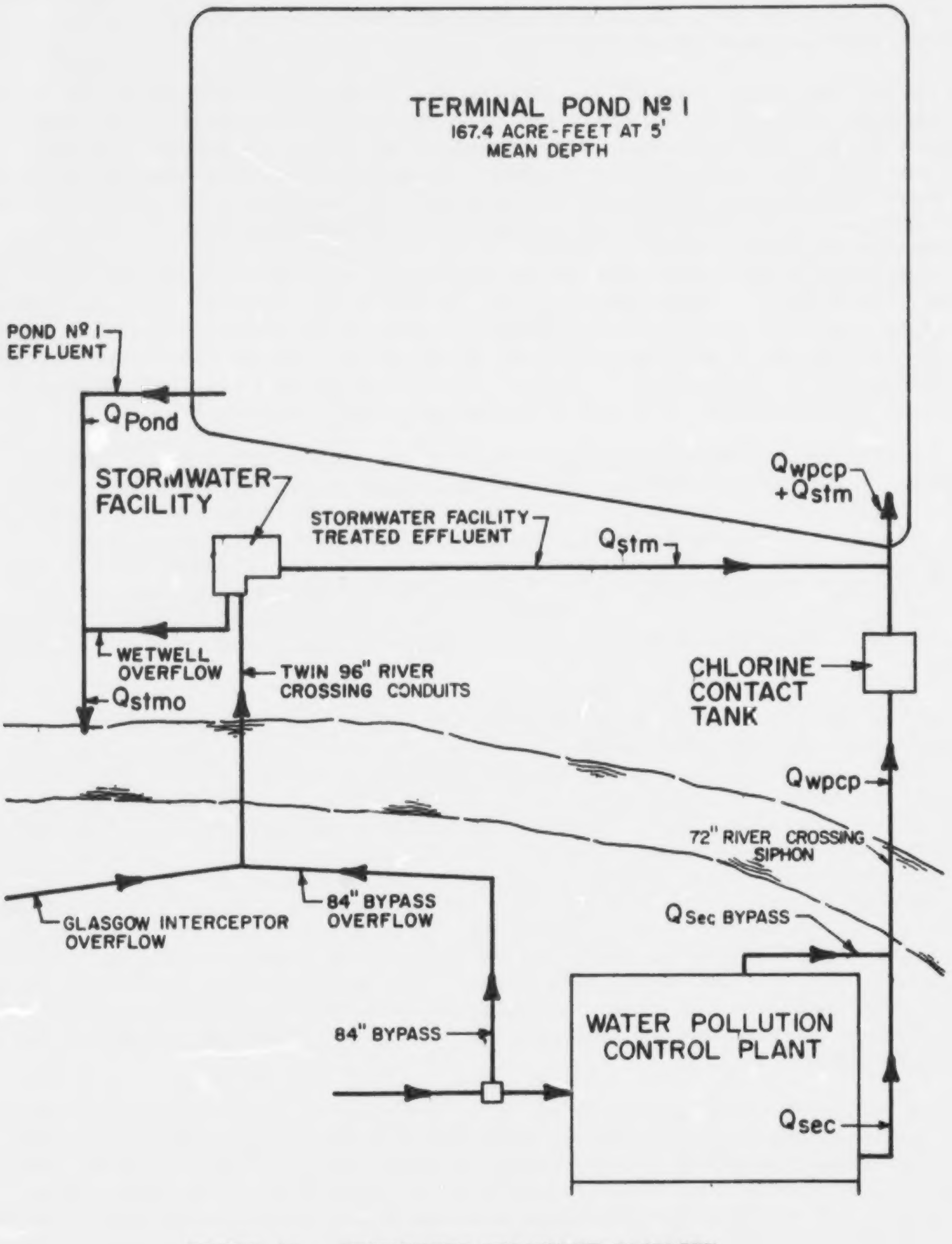

TERMINAL POND Nº 1
167.4 ACRE-FEET AT 5' MEAN DEPTH
POND Nº 1 EFFLUENT
Q_{Pond}
STORMWATER FACILITY
STORMWATER FACILITY TREATED EFFLUENT
Q_{stm}
$Q_{wpcp} + Q_{stm}$
WETWELL OVERFLOW
Q_{stmo}
TWIN 96" RIVER CROSSING CONDUITS
CHLORINE CONTACT TANK
Q_{wpcp}
72" RIVER CROSSING SIPHON
GLASGOW INTERCEPTOR OVERFLOW
84" BYPASS OVERFLOW
Q_{Sec} BYPASS
84" BYPASS
WATER POLLUTION CONTROL PLANT
Q_{sec}

FIGURE 42 - STORMWATER TREATMENT FACILITY FLOW ROUTING SCHEMATIC DIAGRAM

It was reasoned that to divert this overflow to primary treatment and detention in a holding basin would appreciably decrease the amount of raw untreated overflow passing to the river.

One of the key objectives of the program was to determine the shock and total pollutional loading to the river for each significant rainfall event from both treated (i.e., post-construction) and untreated (i.e., pre-construction) sources that were part of this project. To assess the shock loading of treatment provided by the Stormwater Facility and the terminal pond on these overflow quantities, a longitudinal hydrographic analysis was made of several stormwater treatment events. Figure 43 is the hydraulic mass flow hydrograph for the longitudinal flow through the treatment process drawn in the schematic flow diagram. From this diagram, it should be apparent that the hydraulic flow response for this event indicates that an accumulation of 0.15 inch (0.38 centimeter) of precipitation was required to trigger the 84-inch bypass diversion to the Stormwater Facility. This accumulation required about 2 hours to accumulate at this precipitation rate for this event.

As the stormwater-induced flow was received at the Water Pollution Control Plant, the raw sewage pumping rate was increased to keep up with the rising interceptor levels. It took an additional 30 minutes to accumulate enough in the interceptor sewers to start filling the Stormwater Facility wetwell. Ninety minutes later, the wetwell water surface reached a depth of 26.0 feet (7.92 meters). The 50 MGD pump was started and began pumping down the wetwell. At this moment, the Stormwater Facility began operations as a treatment process with these flows going to the screening units and then to the terminal pond.

As can be verified from Figure 43, the pond inflow, as represented by the summed flows of the Water Pollution Control Plant effluent, Q_{WPCP}, plus the Stormwater Facility flow, Q_{stm}, shows a peak flow rate of 77,000 gpm (292,000 liters per minute) at approximately 30 minutes after the 50 MGD stormwater pump began operating. This flow corresponds to a 100 percent increase over the instantaneous raw sewage pumping rate of the plant alone. In other words, the stormwater pump has essentially doubled the instantaneous peak flow rate to the pond by taking the diverted raw sewage overflow through the screening facility. This amount of raw, untreated wastes would have been bypassed to the river directly starting with the 84-inch bypass flow at 2:00 a.m. if the Stormwater Facility had never been built. Furthermore, this flow might have began overflowing to the river at the point where the overflow elevation for the wetwell was exceeded at about 4:00 a.m. if the stormwater pumps had not been operated.

Further examination of Figure 43 reveals that the terminal pond itself had a noticeable effect on the hydraulic flow variable associated with this event. The pond began receiving the Stormwater Facility effluent at 4:00 a.m. The peak inflow already described arrived shortly thereafter. The pond volume essentially absorbed this step input flow variable, dampened the peak flow by its available volume, and allowed this flow to pass through gradually. The peak flow for the pond effluent is shown to be at 8:00 a.m. on the 15th. This peak flow was computed to be 50,300 gpm (190,000 lpm) representing a 28 percent increase over the instantaneous raw sewage pumping rate at that time.

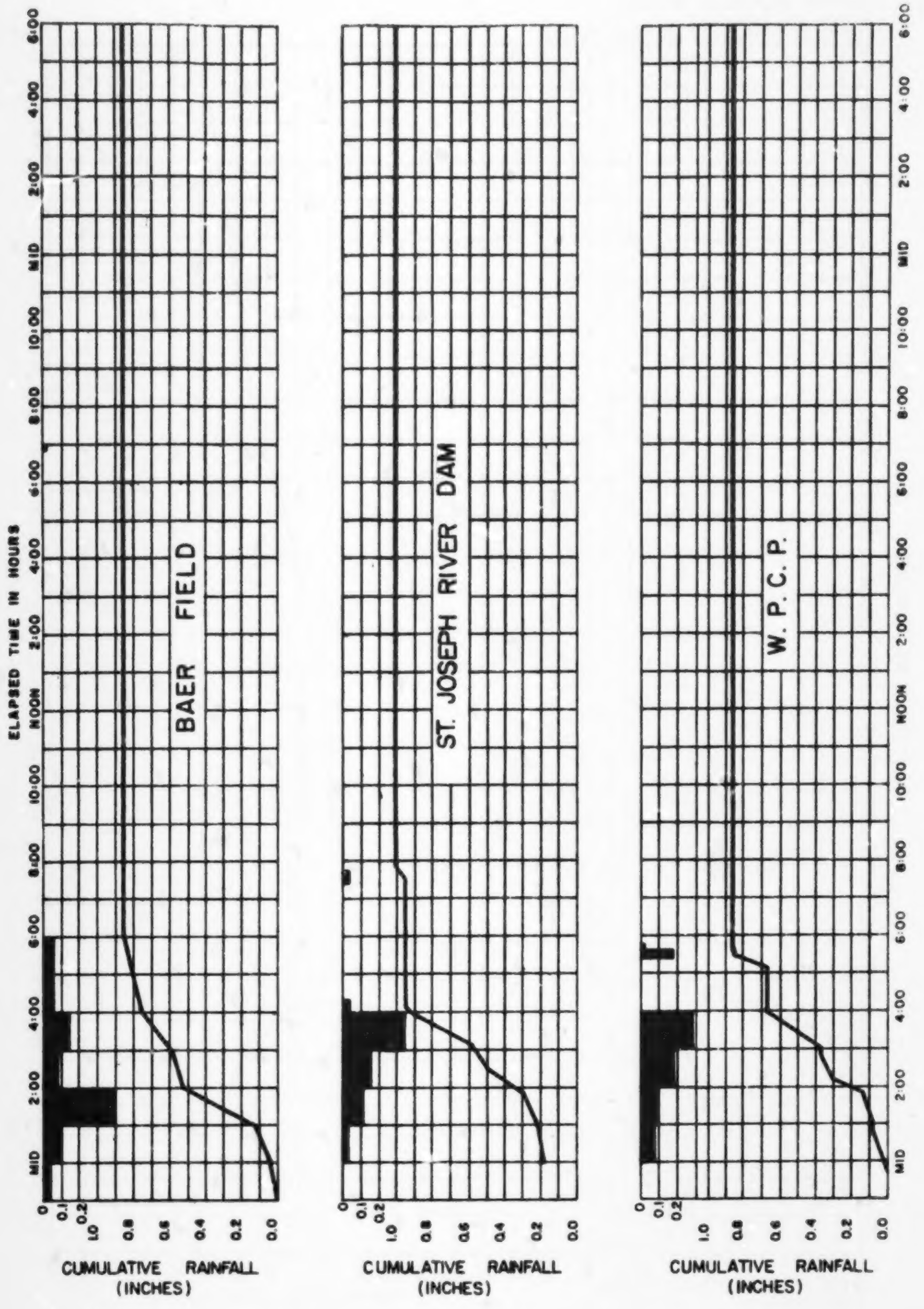


FIGURE 43 - RAINFALL HYDROGRAPH AND LONGITUDINAL HYDRAULIC FLOW IN MGD - PART 1

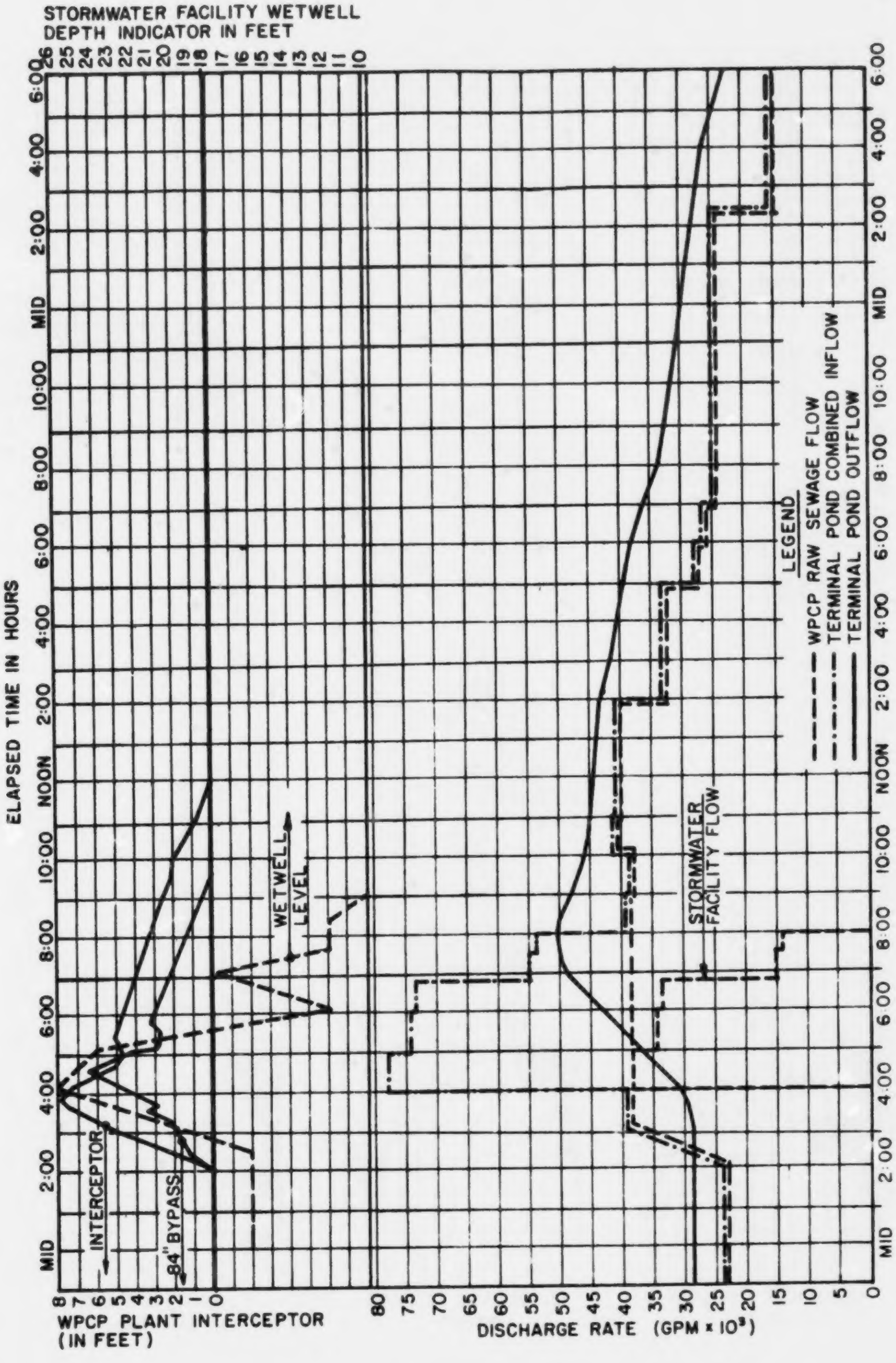


FIGURE 43 - RAINFALL HYDROGRAPH AND LONGITUDINAL HYDRAULIC FLOW IN MGD - PART 2

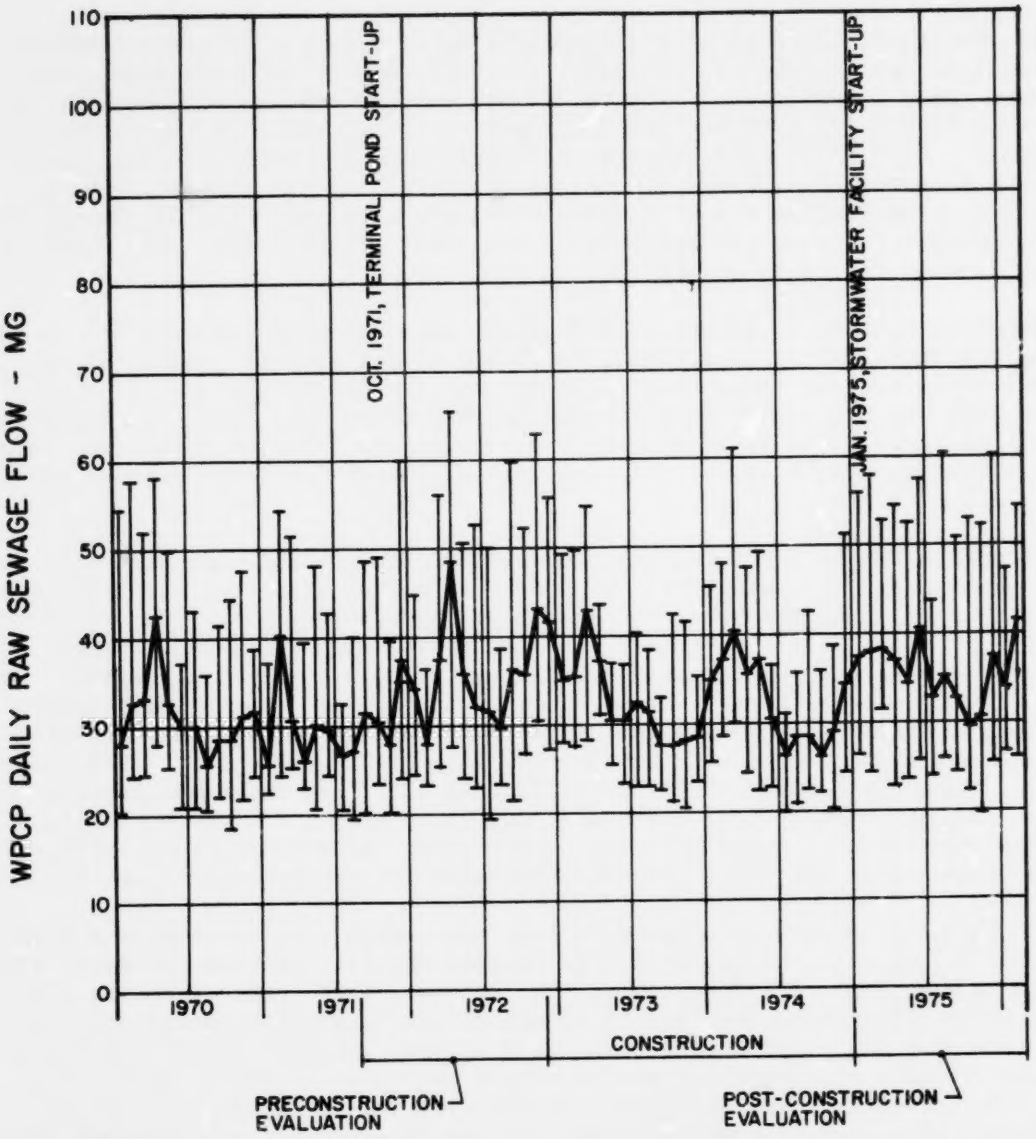
Furthermore, 7.1 million gallons (27 million liters) were the volume of stormwater treated and pumped which might otherwise have passed on to the river untreated. Starting at 2:00 a.m., this flow was protracted in its discharge rather than peaking at about 4:00 a.m.

This result is judged to be a significant benefit in and of itself because it shows that the Stormwater Facility plus the treatment pond can essentially reduce the short-term peak flow to the stream from the 84-inch bypass by as much as 55,000 gpm (208,000 lpm) (maximum discharge capacity for the 84-inch bypass). This would have amounted to about 5 percent of the average daily stream flow for this date. Instead of receiving the raw stormwater bypass directly, the treatment pond reduced this peak loading to 10,900 gpm (41,000 lpm) coming at a time six hours later when most of the rest of the short-term peak flows had already passed.

From observations of the appearance of the terminal pond effluent and those sampling data that are available, there was no evidence of pond treatment short-circuiting or other hydraulic impairment of pond treatment efficiency attributed to the inflow of screened combined sewer overflows. Wind and wave action were observed to muddy the pond contents on occasion. However, there did not appear to be any correlation with stormwater overflow pumping and pond effluent turbidity.

In addition to the short-term hydraulic flow response given in Figure 43, a long-term plant raw sewage flow analysis was performed on the plant operating data available for the period from January, 1970 to February, 1976 as shown in Figure 44. Plotted in this figure are the mean, minimum and maximum monthly raw sewage pumping data available from plant operating records. Note that the post-construction period flows do not appear to be significantly different from the long-term trends recorded for the plant from the period preceding start-up of the Stormwater Facility in January, 1975. This information is not considered significant except to note that the peak flow days for 1975 plotted on this figure exceed 50 MGD (2.10 cumec) for eleven of the twelve months in this period. In the preceding three years, there were only eleven days when the plant raw sewage pumping rate exceeded this amount.

This higher frequency of maximum 24-hour raw sewage pumping rate is a consequence of the construction of the Stormwater Facility because the plant interceptors must fill to a depth showing at 3.5 feet (1.07 meters) on the plant bypass overflow depth recorder to divert to the Stormwater Facility. This has improved plant operations from the standpoint of treatment of more raw wastes because the operators are much more conscious of water depth levels in the plant interceptors. Formerly, if the overflow elevation was exceeded briefly, an amount of combined sewer raw untreated wastes was bypassed without much attention. After completion of the facility, the operators are required to alert the plant superintendent that some flow was diverted to the Stormwater Facility which requires some form of pumping and clean-up at a minimum. The resultant has been to use the available storage in the interceptors up to 3.5 feet (1.07 meters) depth on the indicator, increase the plant raw sewage pumping rate to maintain less than 3.5 feet (1.07 meters) until the maximum rate is reached, and continue to pump up to a depth of 6.0 feet (1.83 meters) where the Stormwater Facility wetwell overflow is exceeded. This has usually



Note: 1 MGD = 0.0438 cumec

FIGURE 44 - LONG-TERM PLANT RAW SEWAGE PUMPING DAILY HYDRAULIC FLOW IN MGD

resulted in much less frequent raw sewage overflows because the time of the peak flow depth at the WPCP has been greatly lengthened by this interceptor storage strategy.

Dissolved Oxygen--

Since the treatment pond was shown to be able to absorb the inflow of diverted screened combined sewer overflow without apparent short-circuiting, the routine daily pond effluent sampling program results were analyzed for evidence of pond performance deficiencies that may have resulted from the addition of combined sewer overflow pumped and screened at the Stormwater Facility. The terminal pond effluent data were separated into wet weather and dry weather periods for both the pre-construction and post-construction periods. The wet weather days were identified as those days where the plant 84-inch bypass diversion recorded a diversion of raw sewage through this bypass conduit for both the pre- and post-construction phases. When the post-construction data were analyzed, the day following such an event was also included in the wet weather category.

Table 24 is the presentation of the best-fit normal distribution on the data available from daily routine plant monitoring records. As can be verified by examination of these data, there is no perceptible effect on the pond effluent dissolved oxygen that can be attributed to the addition of storm-caused screened overflows to the terminal pond.

Table 24. WATER POLLUTION CONTROL PLANT TERMINAL POND EFFLUENT DISSOLVED OXYGEN

	Pre-Construction Evaluation Phase	Post-Construction Evaluation Phase	
	All Days	Wet Weather Days Only	All Days
Arithmetic Mean Pond Dissolved Oxygen (mg/l)	4.06	3.80	3.55
Standard Deviation (mg/l)	0.81	1.30	1.35
Number of Observations	375	136	332
Range of Values Observed (mg/l)	1.8 to 9.7	1.5 to 9.2	1.5 to 15.8

Another aspect of the operation of the terminal pond is the effect of the stormwater on the maximum and minimum values of the pond effluent dissolved oxygen. In one aspect, Table 24 represents the condition where the pond effluent variation is shown probabilistically. Examination of the pond effluent dissolved oxygen over a longer term lends a slightly different view to this trend.

Figure 45 shows the long-term WPCP plant terminal pond effluent D.O. variation going back to 1970. When the terminal pond went into operation in October, 1971, the pond originally responded gradually by showing an increasing trend of effluent D.O. starting in May, 1972, peaking at a mean value of 8.7 mg/l in December, 1972. This rising mean value was accompanied by consistently rising maximum D.O. levels. Minimum D.O. samples analyzed follow the general upward mean trend although the range of variation possible for these minimum values is less than for either the mean or the maximum.

After this rapid rise from a mean D.O. of 2.8 mg/l in May, 1972 to a mean D.O. of 8.7 mg/l in December, 1972, there followed an equally rapid decline in pond D.O. levels to a mean D.O. low of 3.7 mg/l in July, 1973. Thereafter, there appear to be seasonal rises and falls within the range of mean D.O. values of 2.1 mg/l to 5.2 mg/l through 1975.

Part of the significance of the dramatic rise in pond D.O. for the first year of operation as a plant effluent terminal pond lies in the likely algae "bloom" of the summer and autumn of 1972. This bloom of oxygen-producing algae could be partly responsible for the rise in pond effluent D.O. Indeed, examination of the pond effluent suspended solids data for the same time period tends to support this conclusion by exhibiting a seasonal increase in effluent suspended solids in the warm weather months of July through September or sometimes October. Peak suspended solids values during this period tend to also indicate an increase in some days' pond suspended solids data.

On the other hand, the pond suspended solids data also show high mean values outside of the warm weather period. In fact, the highest monthly mean suspended solids data for the pond effluent came in April, 1972, March, 1973, and February, 1974. Corresponding peak suspended solids naturally followed the mean monthly trend. Minimum values also were consistent with the mean monthly trend.

Let it suffice to say that the addition of Stormwater Facility effluent to the pond did not appear to adversely affect the range or mean values of the pond dissolved oxygen, except to note that the lowest monthly mean value for pond dissolved oxygen occurred in October, 1975 at 2.1 milligrams per liter. In any case, the monthly mean pond effluent D.O. cannot be said to have caused the increase in average values for stream D.O. nor the demonstrated decrease in numbers of days of stream D.O. less than 4.0 milligrams per liter.

Biochemical Oxygen Demand--

One would expect the terminal pond biochemical oxygen demand (BOD_5) values to show essentially the same trend as the dissolved oxygen data. The mean pond D.O. data cited do not show any significant variation in the D.O. concentrations measured in the pre-construction data as compared to the post-construction evaluation period (See Table 24).

Table 25 displays the BOD_5 data results for the plant terminal pond before the introduction of stormwater and after the introduction of stormwater. Figure 46 gives the normal curve display of these data for comparison with the tabulated results.

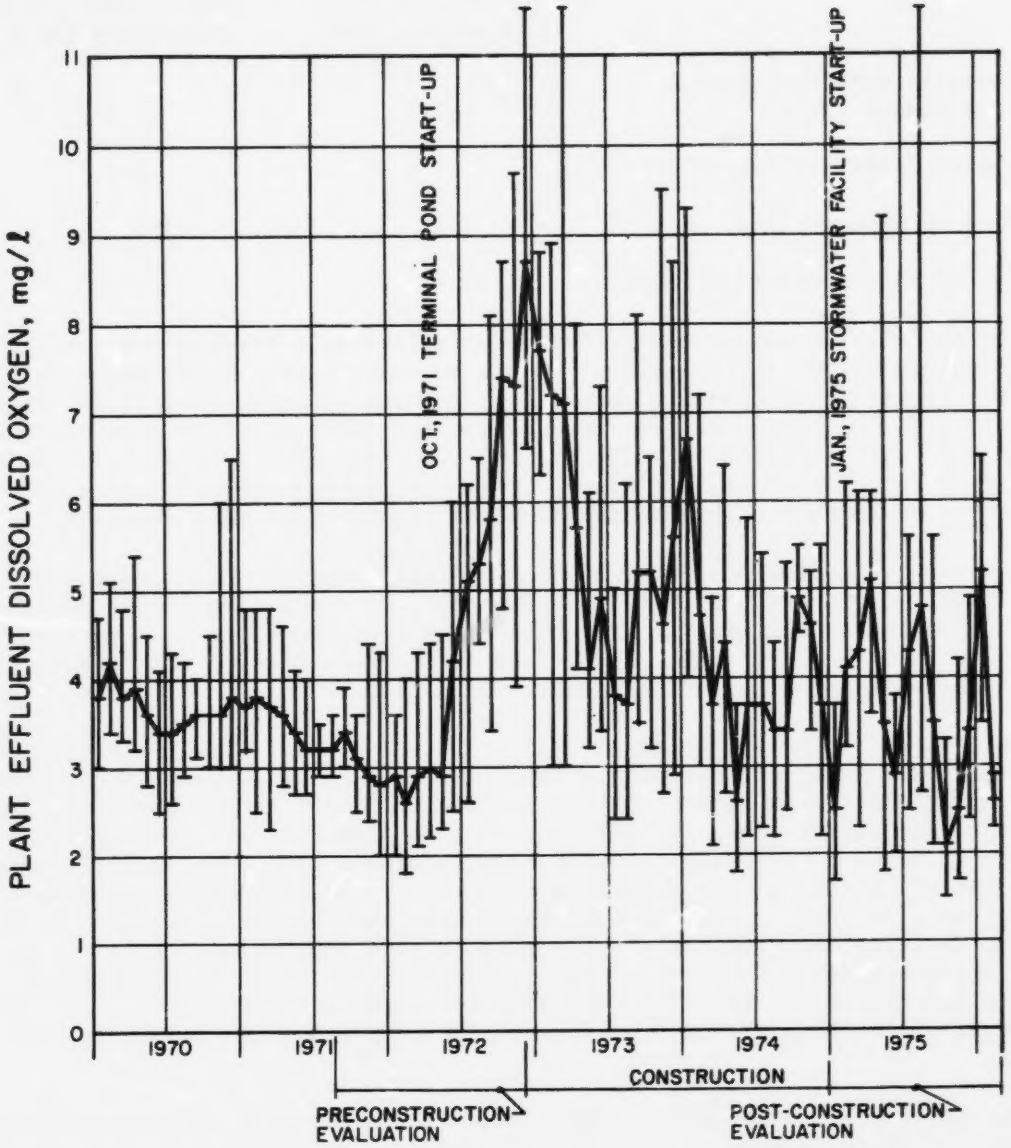


FIGURE 45 - WATER POLLUTION CONTROL PLANT EFFLUENT DISSOLVED OXYGEN LONG-TERM MEAN MONTHLY D.O. CONCENTRATION

Table 25. PLANT TERMINAL POND EFFLUENT
BIOCHEMICAL OXYGEN DEMAND

	Pre-Construction Evaluation Phase	Post-Construction Evaluation Phase
Arithmetic Mean Biochemical Oxygen Demand (mg/l)	13.5	11.5
Standard Deviation (mg/l)	7.5	4.5
Number of Observations	463	392
Range of Values Observed (mg/l)	1 to 52	1 to 28

While there is no measurable difference statistically between the arithmetic mean values cited in Table 25, it should be noted that the two distributions do not appear to fall on the same line. The two standard deviations noted in Table 25, approximated from the normal distribution curve, show different trends. The explanation of this result is not apparent from examination of the data for the two periods cited. Figure 46 is a graphical presentation of these two probability distributions for the terminal pond effluent.

The degree of difference between the two distributions is decidedly and noticeably different. Each distribution is very close to an ideal probability distribution as determined from the raw data (not shown). It is interesting to note that there is such a significant difference between the two distributions shown on the basis of concentration alone.

Upon examination of the terminal pond effluent data for a long-term period of time, a very interesting trend is apparent. Figure 47 shows the long-term plant effluent terminal pond data from January, 1970 through February, 1976. In this six-year period, the effluent BOD_5 is shown to vary greatly from month to month in the range of mean monthly values from 5 to 36 milligrams per liter. Since the pond start-up in October, 1971, the effluent mean BOD_5 has tended to peak in the cold weather months and then suddenly drop at the start of warmer spring weather to a minimum pond BOD_5 value. This tendency is very interesting because it tends to confirm the trend of temperature-dependent biological kinetics for oxidation ponds for the entire period of time prior to the post-construction evaluation period starting in January, 1975.

After January, 1975, note that the trend of terminal pond effluent BOD_5 data tends to remain more stabilized in effluent quality in terms of mean, maximum and minimum values of effluent BOD_5. These results might be attributed to the addition of large volumes of warmer storm flow to the pond in cold weather, tending to keep the pond temperature higher on the average. However, examination of the pond temperature data available (taken at the effluent structure) does not tend to support or discredit this explanation.

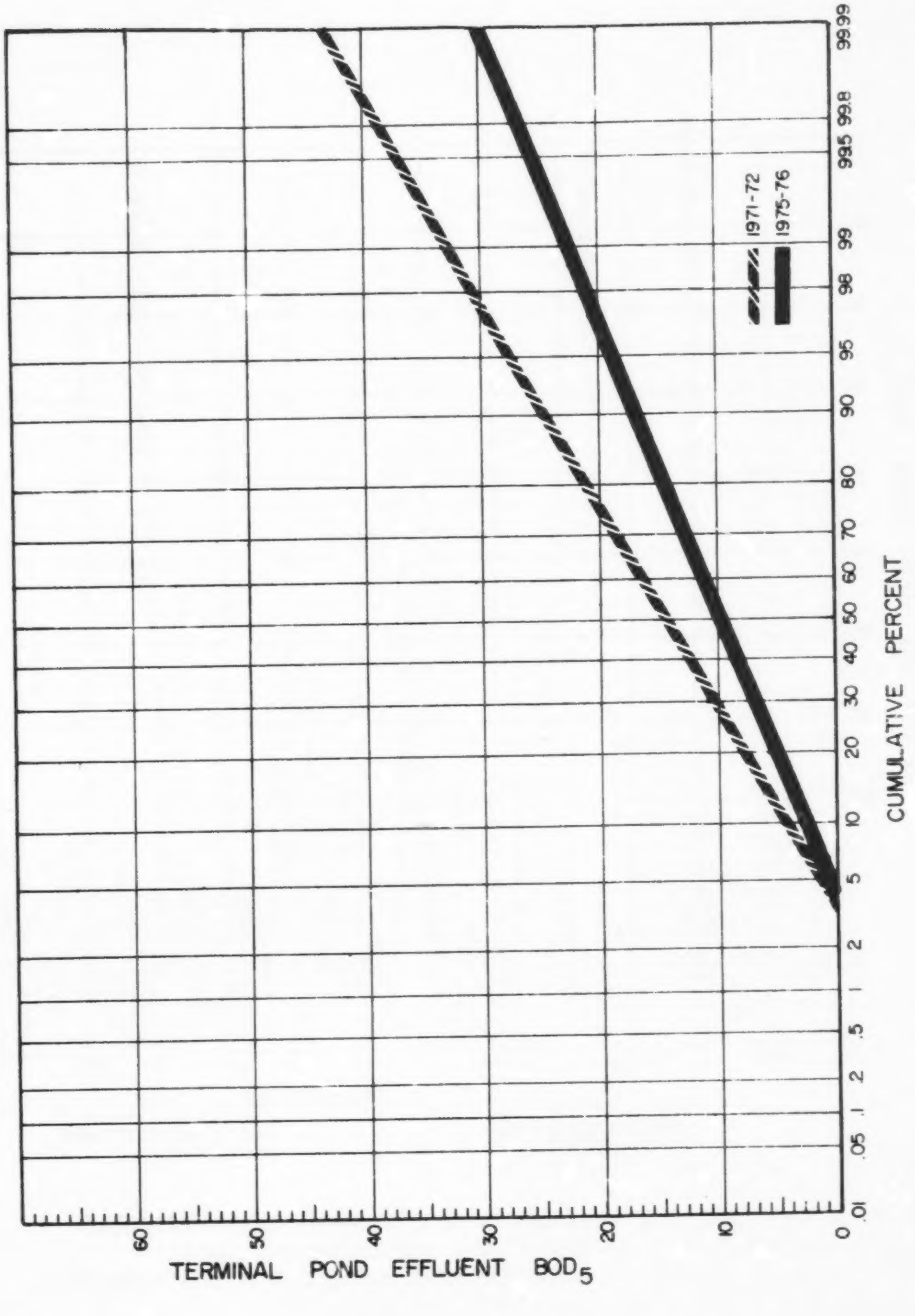


FIGURE 46 - PLANT TERMINAL POND EFFLUENT BIOCHEMICAL OXYGEN DEMAND

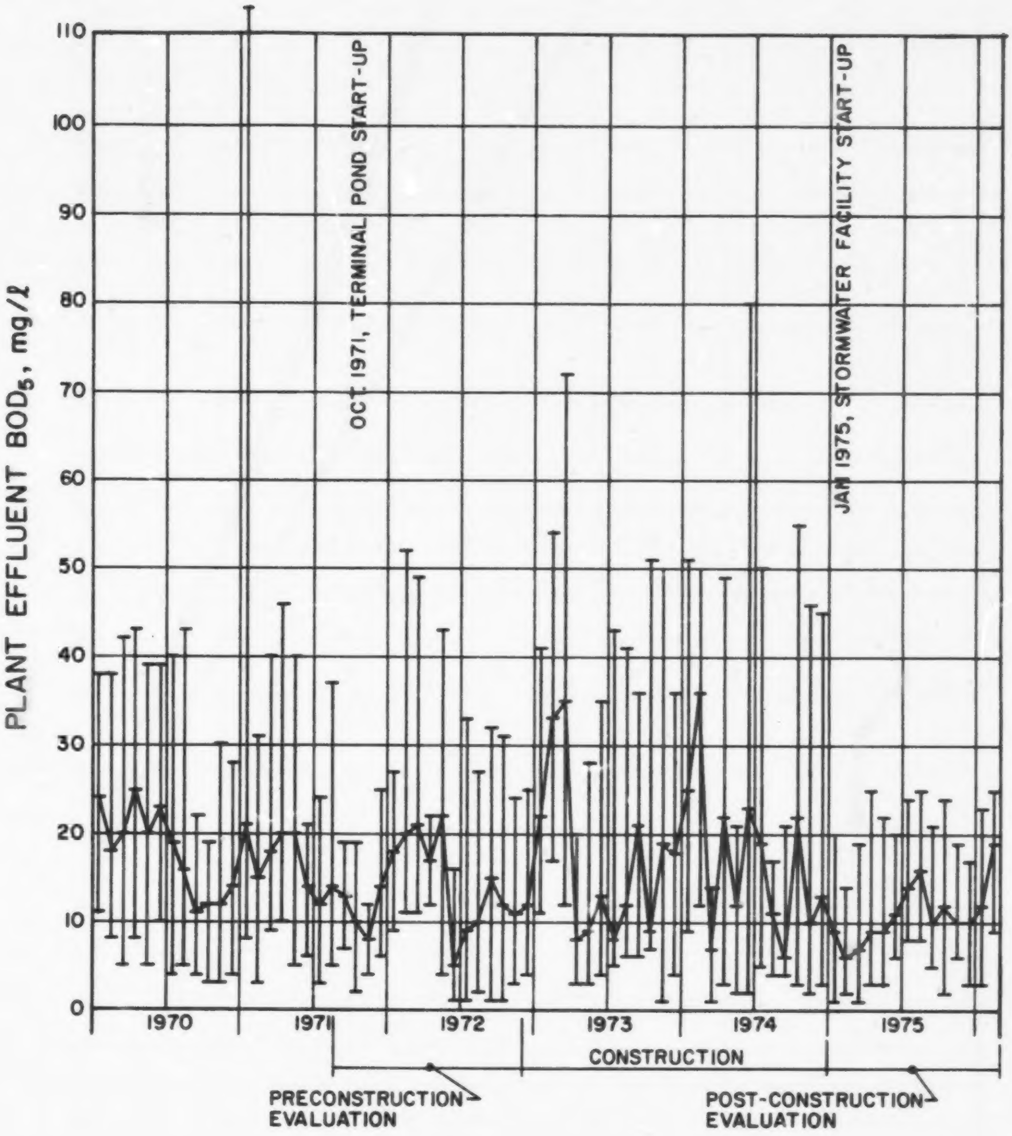


FIGURE 47 - WATER POLLUTION CONTROL PLANT EFFLUENT BIOCHEMICAL OXYGEN DEMAND (BOD5) LONG-TERM MEAN MONTHLY BOD5 CONCENTRATION

The more universal trend of increase in the mean pond effluent BOD_5 during the warm weather months does not seem deserving of an explanation because the post-construction trend seems consistent with the preceding years. However, the lesser range of pond effluent values in 1975 deserves more comment. In previous years, the mean effluent values began an upward trend in July, August and September. The corresponding maximum effluent BOD_5 values appeared to come in this same time period. In contrast to this rising mean, greater variation trend, the 1975 data show a rising mean, steady variation trend in terms of maximum effluent BOD_5.

Table 26 below shows the relationship between the pre-construction and post-construction phase plant secondary treatment effluent BOD_5 for the same time periods.

Table 26. PLANT SECONDARY TREATMENT EFFLUENT BIOCHEMICAL OXYGEN DEMAND

	Pre-Construction Evaluation Phase	Post-Construction Evaluation Phase
Arithmetic Mean Biochemical Oxygen Demand (mg/l)	15.33	29.48
Standard Deviation (mg/l)	9.16	20.15
Number of Observations	483	399
Range of Values Observed (mg/l)	1 to 47	0 to 102

From the comparison of the two sets of data, it is evident that the terminal pond efficiencies of removal based on BOD_5 have improved measurably since the completion of the Stormwater Facility. There are two explanations that come to mind. First, the plant flow has increased over this period somewhat without a corresponding increase in plant capacity. As the plant flow increased, it follows that the secondary treatment effluent concentration should increase somewhat because there has not been a corresponding increase in secondary treatment efficiency over the same time span. In fact, the plant secondary effluent in 1975-76 was substantially higher than the 1971-72 data. This fact tends to support the contention that the pond efficiency has improved since the addition of stormwater.

On the other hand, the quantity of flow treated by primary treatment and bypassing secondary treatment definitely increased as a consequence of the construction of the Stormwater Facility. In the fourteen-month period of the pre-construction evaluation, there were 1,954 million gallons (7,400,000 cubic meters) total secondary bypass flow sent to the terminal ponds. In the thirteen-month post-construction evaluation period, there were 3,180 million gallons (12,040,000 cubic meters) of primary effluent that did not go through secondary treatment at the Water Pollution Control Plant. In other words, the construction of the Stormwater Facility has prompted an increase in the amount of raw wastewater treated at the Water Pollution Control Plant to the limit of the treatment capacity available.

This contention is also supported by the data obtained in this program. The plant secondary bypass flow is combined with secondary treatment effluent before sampling at the Water Pollution Control Plant. This means the additional organic loading in the primary treatment effluent bypassing conventional secondary treatment is integral to the composite samples obtained throughout this program.

In spite of the much higher pond influent organic loading in the post-construction period when compared to the pre-construction period, the terminal pond effluent was shown to be quite stable even with the addition of raw, chlorinated stormwater. This result was unexpected. Several hypotheses were examined in detail to attempt to explain the apparent higher treatment efficiency for the ponds when loaded with higher organic loadings.

The result was first thought to be a consequence of the slightly shorter average pond hydraulic detention period resulting from the addition of large amounts of stormwater. Event 186 is a typical example of a relatively long duration storm event (See Figure 43). Several other events were shown to have a short-term increase in flow up to 50 percent more storm-caused flow than the Water Pollution Control Plant effluent flow. It was hypothesized that this higher short-term flow would tend to shorten the pond hydraulic detention time sufficiently to result in wash-out of the motile forms of algae and higher forms of aquatic plants present. At the same time, the greater soluble loading introduced in the ponds was stimulating the growth of bacterial forms which tended to settle out in layers of sediments.

Examination of the pond effluent for biological classification of microbiological organisms was not performed because of project time and budget constraints. Nevertheless, results obtained from physical and chemical analyses of pond benthic deposits and effluent wastewater samples tend to support the contention that algae and other motile forms were prevented from taking over the pond by the short hydraulic detention period available. Volatile solids content of pond benthic deposits showed these solids had physical and chemical characteristics to primary settled raw sludge. This tends to confirm the hypothesis that microbiological activity is responsible for the organic pollutant removals observed for the ponds.

The result of the apparent beneficial effects of the addition of raw screened stormwater overflow was so intriguing to the investigators that a more detailed examination of the pond loadings seemed advisable. At first it was hypothesized that there may be some kind of toxicity effect at work in the pond that tended to suppress the effluent BOD_5 concentration. This hypothesis was advanced after the examination of the long-term mean month pond effluent COD trends shown in Figure 50 following.

Since the data base from which these data were taken was shown to be consistent over time, an alternative hypothesis stemming from the examination of the pond effluent suspended solids was advanced. In this alternate hypothesis, the pond was supposed to be responding to the Stormwater Facility BOD_5 load resulting from the screened effluent BOD_5 remaining plus the screening equipment overflow points raw untreated BOD_5 and suspended solids load. However, examination of the long-term terminal pond suspended solids concentrations shows a similar trend for this pollutant parameter.

The investigators decided that an examination of the long-term terminal pond organic and suspended solids loadings was necessary to further attempts to explain these results. Plotted in Figure 48 are the data for the terminal pond mean monthly organic loading rate in terms of pounds BOD_5 per acre per day. These data were correlated with organic removal efficiency and plotted in a bivariate manner using linear paper in a "scattergram", shown in Figure 49.

As can be verified from Figure 48, it appears as though there is a limiting minimum organic loading that can be applied to the pond below which the treatment efficiency as measured by BOD_5 is adversely affected. After the completion of the Stormwater Facility, BOD_5 removal efficiency averaged 75 percent ($\sigma = 4.83$ percent). In the preceding 39 months of operation, the mean monthly organic removal efficiency was 47 percent ($\sigma = 42.3$ percent). Figure 49 shows the bivariate analysis of the pond treatment efficiency as a function of total pond organic loading. Note that there are no negative treatment efficiency results reported for loading rates greater than 200 pounds BOD_5 per acre per day (224 kg BOD_5 per hectare per day). For loading rates in the range of 100 to 199 pounds BOD_5 per acre per day (112 to 223 kg BOD_5 per hectare per day), there are an equal number (7) of monthly mean values greater than 50 percent than less than 50 percent (also 7). For loading rates greater than 200 pounds BOD_5 per acre per day (224 kg BOD_5 per hectare per day), there were only two months with less than 50 percent mean monthly removal efficiency. None of the post-construction evaluation period months (all of 1975 and the first six weeks of 1976) had organic loading treatment efficiencies less than 60 percent.

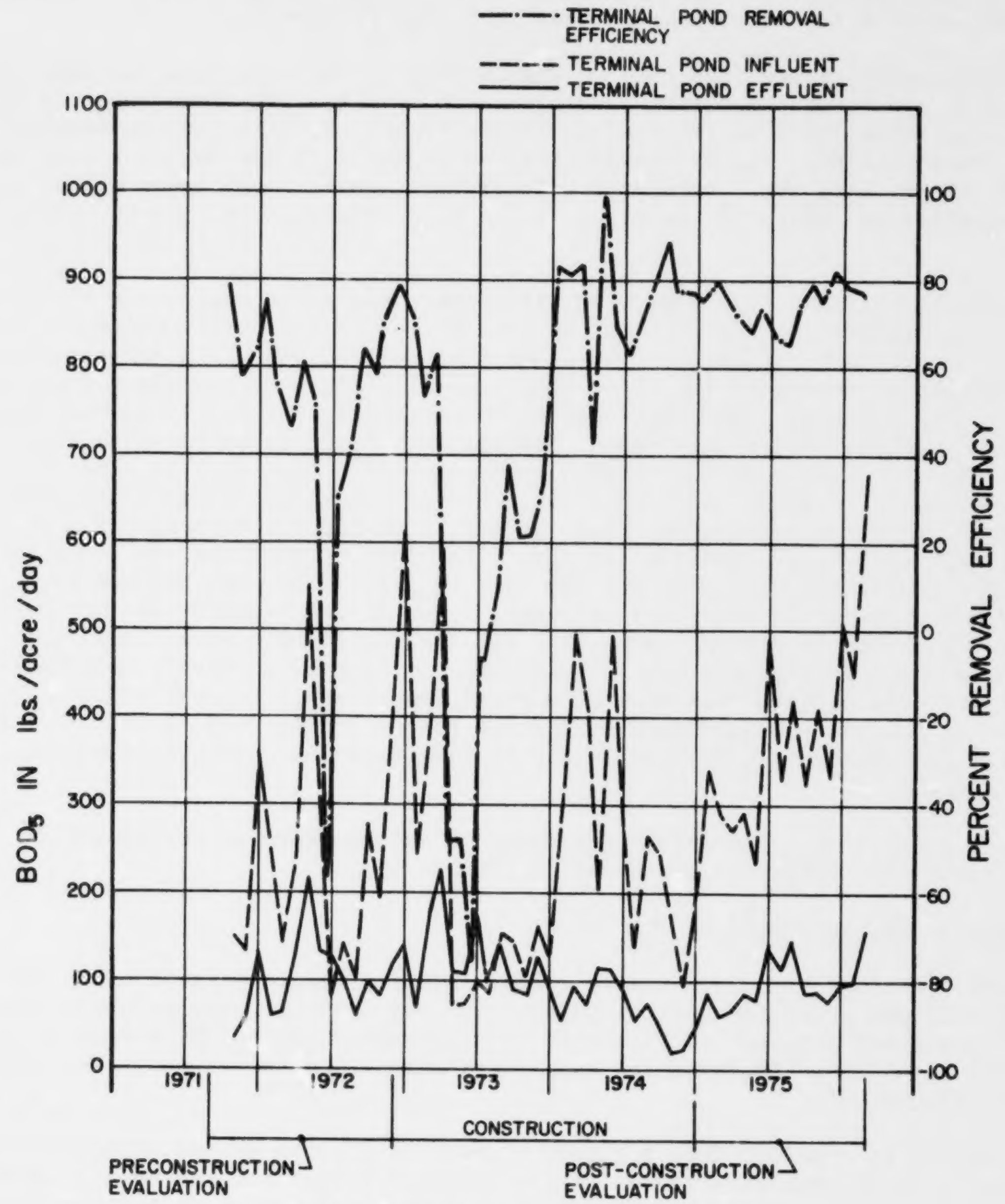
These results tend to confirm the trend of values shown in the long-term terminal pond effluent concentration (Figure 47).

Chemical Oxygen Demand--

Plant operators have been taking chemical oxygen demand data on plant influent and effluent flows for several years. Monthly mean COD results are plotted for the plant effluent in Figure 50 from January, 1970 to the end of 1975. These results are included for comparison with the data for the plant effluent BOD_5 (Figure 47).

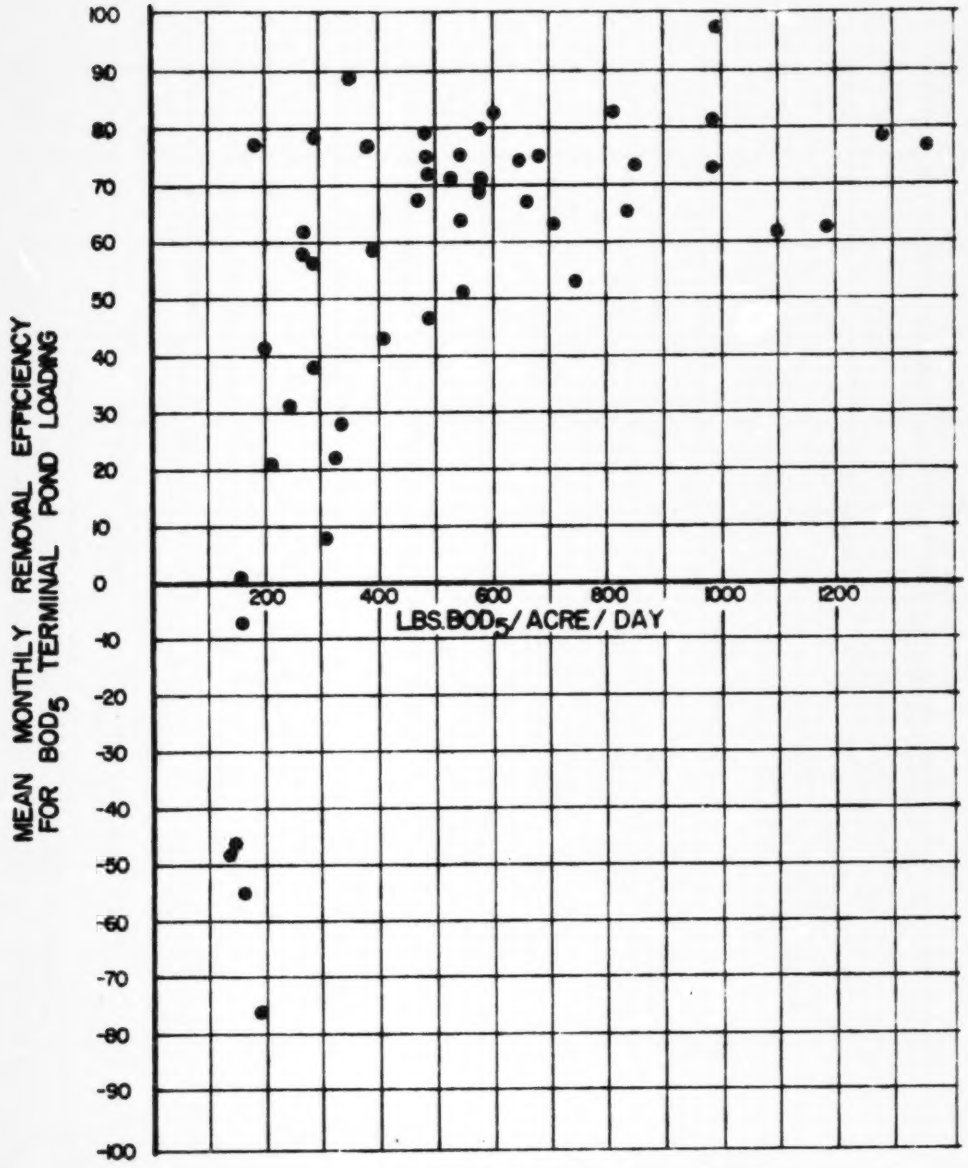
It should be apparent from a glance at each of Figures 47 and 50 that there appears to be some differences in the trend of values for the monthly means. Also, the much greater variation between minima and maxima in the two displays of data tends to obscure any trend that may be observed.

Water Pollution Control Plant COD data are based on the so-called short method procedure, rather than the reflux method. The difference between the BOD_5 and COD data may be due to some organic residuum in the terminal effluent measurable by the COD test method used that does not contribute to five-day biochemical oxygen demand. Whatever this difference is, it must be noted



Note: lbs./acre/day x 1.12 = kg/ha/day

FIGURE 48 - LONG-TERM TERMINAL POND MEAN MONTHLY ORGANIC LOADING RATE



Note: lbs/acre/day x 1.12 = kg/ha/day

FIGURE 49 - TERMINAL POND MEAN MONTHLY BOD_5 LOADING REMOVAL EFFICIENCY AS A FUNCTION OF POND INFLUENT BOD_5 LOADING RATE

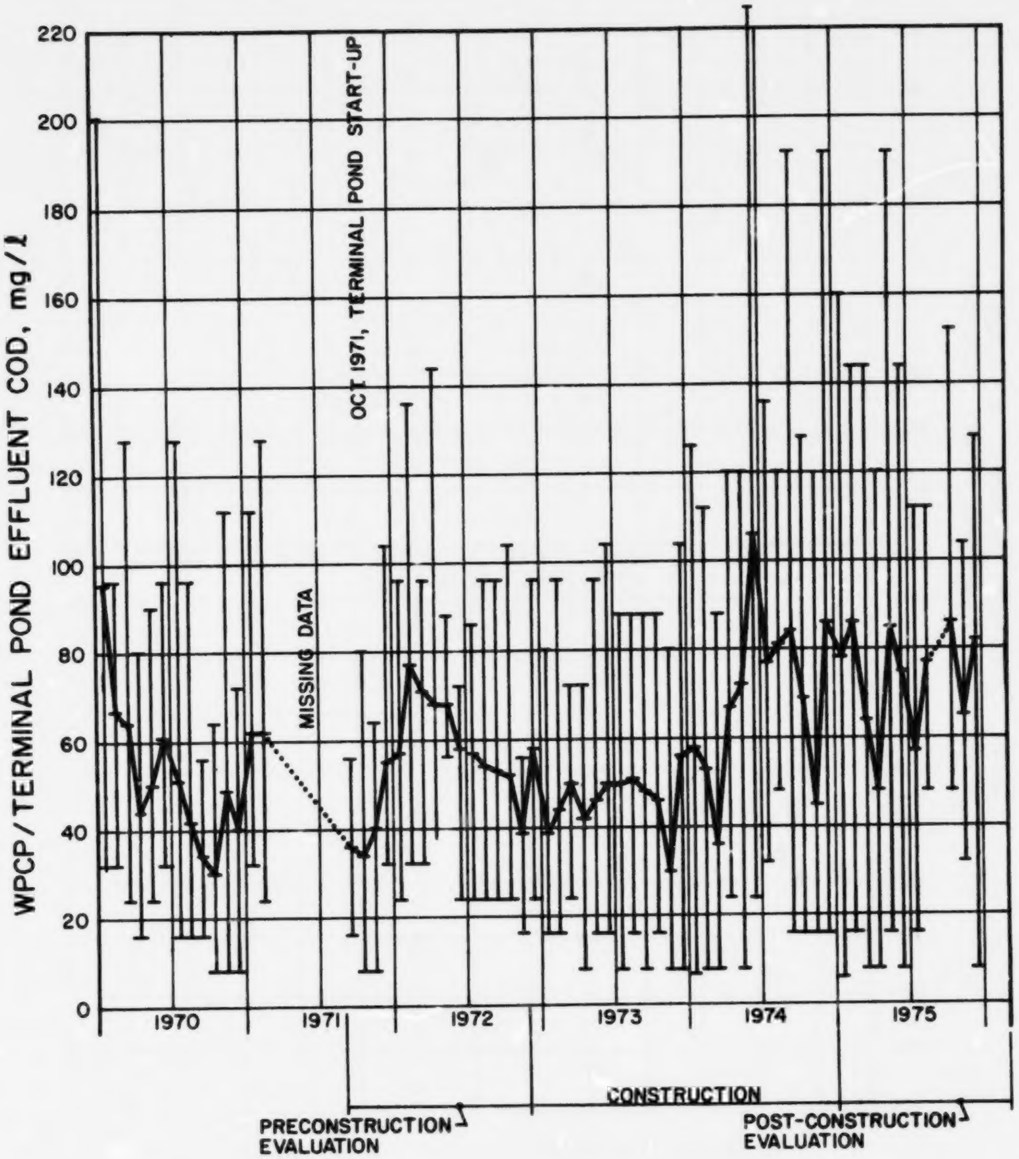


FIGURE 50 - TERMINAL POND EFFLUENT CHEMICAL OXYGEN DEMAND LONG-TERM MEAN MONTHLY CONCENTRATION

that the 1975 mean trend values for COD are generally higher than the years preceding. Whether this increase in trend mean values can be attributed to the addition of stormwater to the terminal pond remains to be seen.

Suspended Solids (Nonfilterable Residue)--

Terminal pond effluent suspended solids appear to follow a similar pattern to BOD_5. The rises and falls in the monthly mean effluent suspended solids correspond to the similar peaks and valleys for BOD_5. The range of values observed in the time prior to the 1975 post-construction evaluation also seem to show a cold weather peak range in mid-winter and another, lesser peak in the warmer mid-summer months.

Figure 51 represents the long-term trend of mean monthly suspended solids data obtained for the terminal pond effluent. A similar trend to that described for BOD_5 exists for the post-construction evaluation period of 1975-1976 in terms of the ranges of monthly maxima and minima experienced. Also, the amount of fluctuation from monthly mean to monthly mean is shown to be much less for the post-construction period than for any time prior to this period. Furthermore, the range of variation of the mean monthly suspended solids measured is shown to be substantially reduced.

Table 27 is an attempt to quantify these observations of the data. The statistics chosen for these data are based on arithmetic means and standard deviations because the normal curve distribution was shown to provide the best fit for the data.

Table 27. PLANT TERMINAL POND EFFLUENT SUSPENDED SOLIDS (NONFILTERABLE RESIDUE)

	Pre-Construction Evaluation Phase	Post-Construction Evaluation Phase
Arithmetic Mean Suspended Solids (mg/l)	20.	11.5
Standard Deviation (mg/l)	31.	11.0
Number of Observations	455	381
Range of Values Observed (mg/l)	2 to 213	1 to 44

The corresponding Water Pollution Control Plant secondary effluent suspended solids is shown in Table 28. These pre-construction and post-construction data contain the plant secondary bypass as well as the secondary treatment effluent.

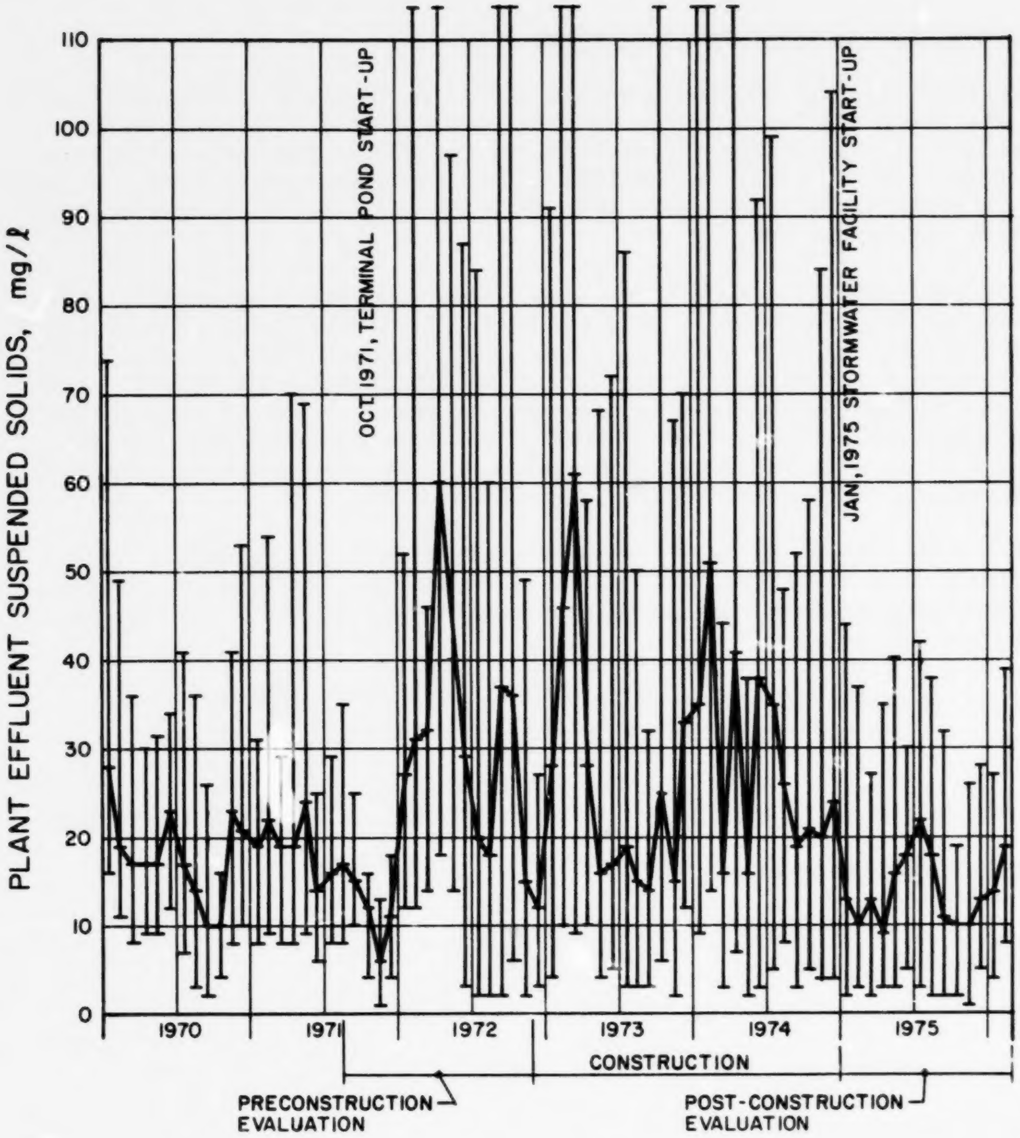


FIGURE 51 - WATER POLLUTION CONTROL PLANT EFFLUENT SUSPENDED SOLIDS LONG-TERM MONTHLY SUSPENDED SOLIDS CONCENTRATION

Table 28. PLANT SECONDARY TREATMENT EFFLUENT SUSPENDED SOLIDS

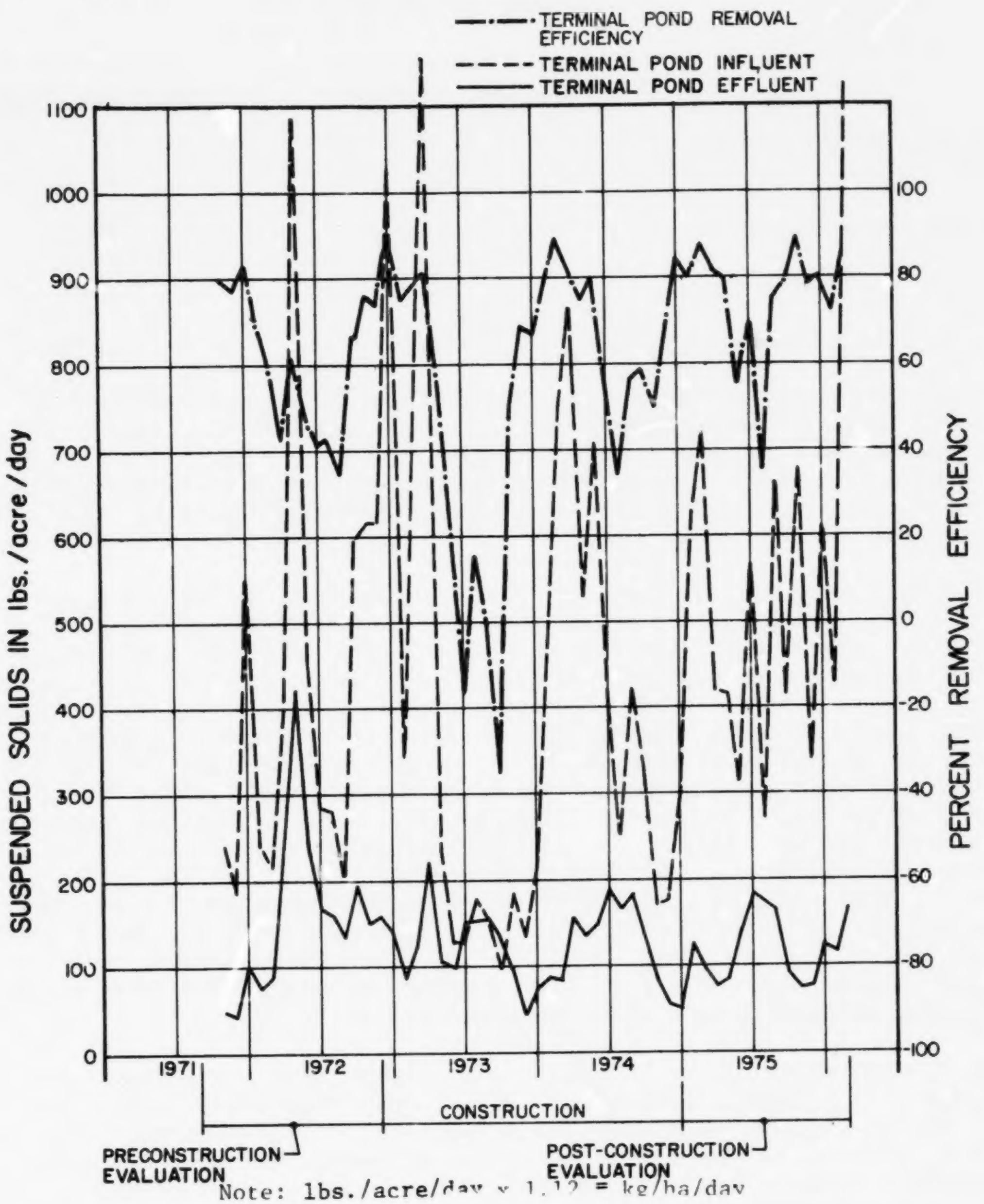
	Pre-Construction Evaluation Phase	Post-Construction Evaluation Phase
Arithmetic Mean Suspended Solids (mg/l)	32.64	32.70
Standard Deviation (mg/l)	26.90	28.26
Number of Observations	453	399
Range of Values Observed (mg/l)	2 to 213	2 to 256

It should be noted that, in contrast to the plant BOD_5 data summarized in Table 25, above, these results do not indicate a measurable difference between pre-construction and post-construction results. This difference is very interesting because the same primary treatment effluent that bypasses secondary treatment is measured in the BOD_5 and suspended solids data. This result is unexplained and is merely noted as such.

A long-term terminal pond suspended solids loading rate analysis was performed on the same data base as was completed for BOD_5. Figure 52 shows the mean monthly variation in terminal pond influent and effluent suspended solids loading rate as pounds suspended solids per acre per day. From these data, it can be shown that the pond effluent loading rate is a very heavily damped response to the influent loading. Furthermore, by comparison of Figure 52 showing the pond effluent mean monthly loading rate with Figure 51 showing the pond effluent mean monthly concentration, it may be shown that for the monthly mean values at least, the pond effluent suspended solids loadings correspond very closely to the effluent suspended solids concentration. Similarly, comparison of Figure 48, pond effluent mean monthly organic loading rate, with Figure 51, pond effluent mean monthly suspended solids concentration, shows a close correlation in the peaks and valleys, particularly in the period since the start-up of the Stormwater Facility. It should be noted that the pond effluent mean monthly COD and NH_4^+-N do not conform to the same trend of values as the pond effluent mean monthly suspended solids.

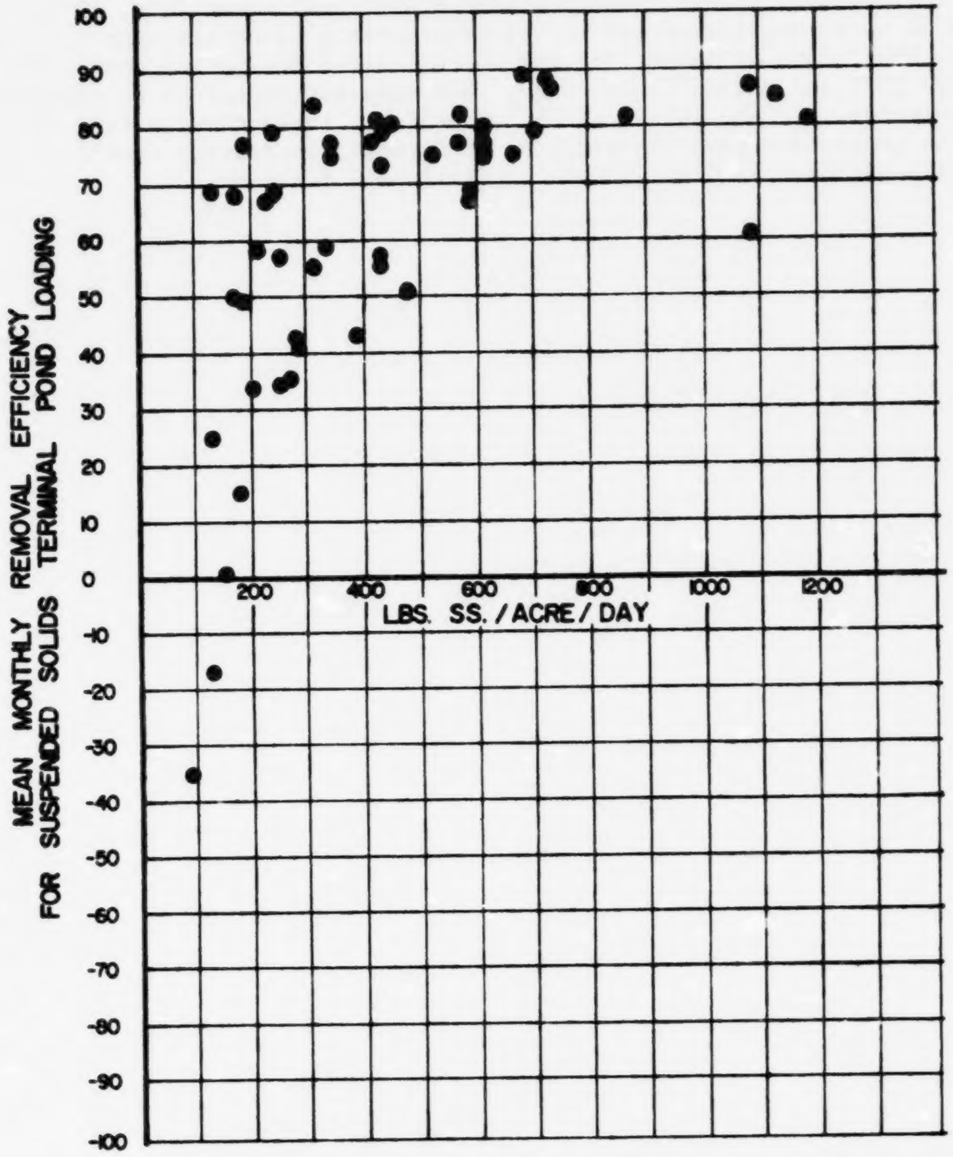
Figure 52 also shows the mean monthly removal efficiency for terminal pond suspended solids loading data for the period since pond start-up in October, 1971. The mean monthly suspended solids averaged 56.9 percent for the period from pond start-up to the completion of the Stormwater Facility (December, 1974) with a standard deviation of 28.3 percent. The corresponding mean monthly suspended solids loading removal efficiency for the 14-month post-construction evaluation period commencing in January, 1976 was 74.8 percent at a standard deviation of 14.0 percent.

Figure 53 is a bivariate "scattergram" of these mean monthly suspended solids loading removal efficiency as a function of the suspended solids loading rate.



Note: lbs./acre/day x 1.12 = kg/ha/day

FIGURE 52 - LONG-TERM TERMINAL POND MEAN MONTHLY SUSPENDED SOLIDS LOADING RATE



Note: Lbs/Ac/day x 1.12 = kg/ha/day

FIGURE 53 - TERMINAL POND MEAN MONTHLY SUSPENDED SOLIDS LOADING REMOVAL EFFICIENCY AS A FUNCTION OF POND INFLUENT LOADING RATE

Total Phosphorus--

Chemical precipitation of phosphate materials in the Water Pollution Control Plant secondary treatment flows has been operational since July, 1973. Figure 54 is a long-term trend of the mean monthly plant effluent total phosphorus. The trend of values for the post-construction period commencing in January, 1975 and ending in February, 1976 does not appear to be significantly different from the months preceding. Therefore, these results tend to support the contention that the addition of screened stormwater overflows to the pond will not adversely affect the terminal pond effluent.

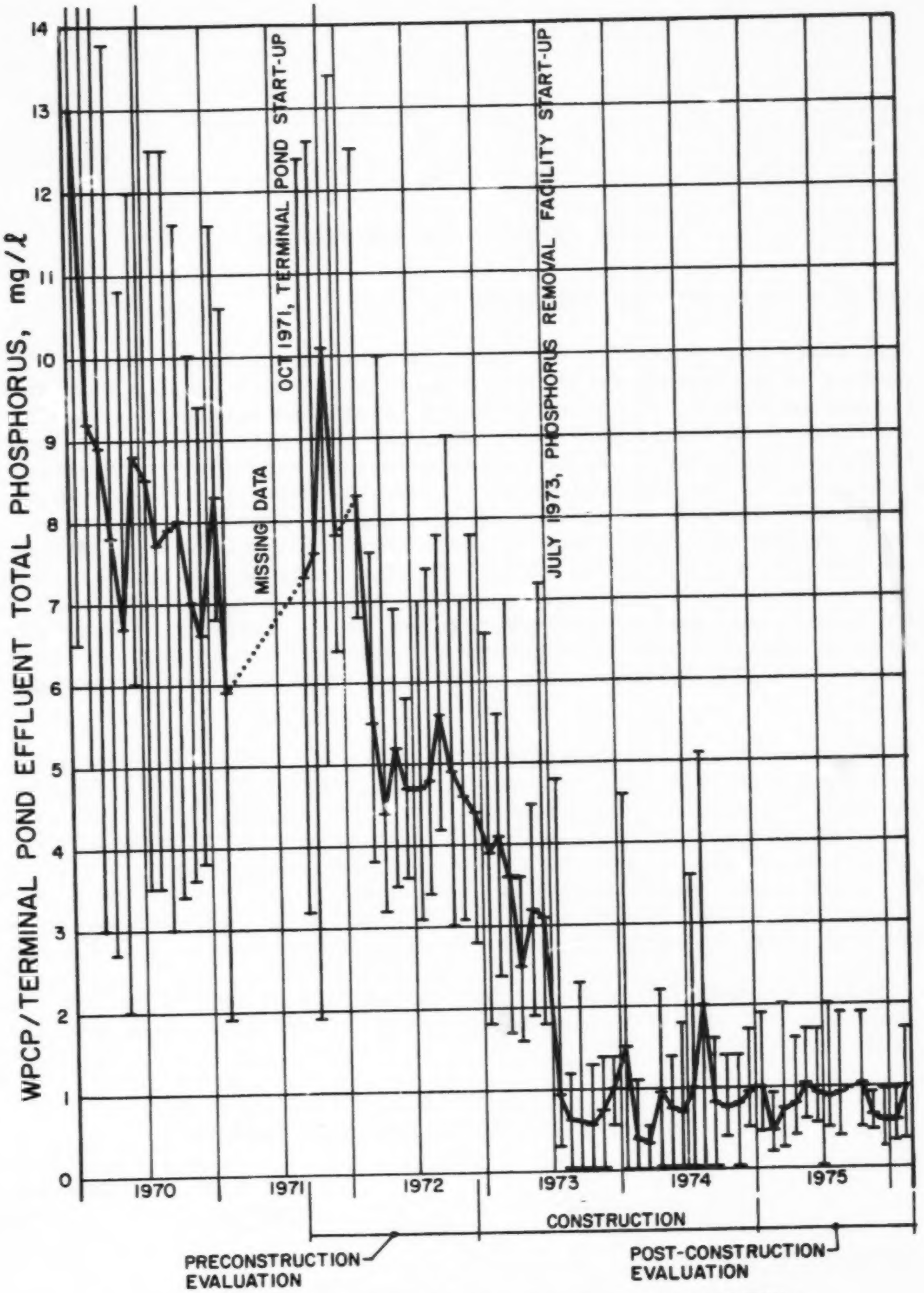


FIGURE 54 - WATER POLLUTION CONTROL PLANT EFFLUENT TOTAL PHOSPHORUS LONG-TERM MEAN MONTHLY CONCENTRATION

SECTION 7

COST ANALYSIS FOR THE PROGRAM

CONSTRUCTION COSTS FOR THE STORMWATER FACILITY

The Fort Wayne Stormwater Demonstration Project construction costs included the cost of constructing the Glasgow Avenue and 84-inch plant bypass regulator overflow sewers, the river crossing sewers and structures, the Stormwater Pumping Facility wetwell and pumps, the Screening Building including its piping and equipment, additional chlorination capacity for the Water Pollution Control Plant chlorination system, the chlorine contact channel at the Stormwater Facility and the associated piping, fittings, electrical connections, structures, appurtenances and conduits to tie the project together. For the purposes of discussion of construction costs of the facility, only the actual construction costs attributed to the pumping, screening and chlorination of combined sewer overflows received at the facility were considered. The costs of constructing the associated sewers and structures were excluded from this evaluation because these costs were determined to be relatively commonplace.

The total approved grant amount for the construction of the project was $1,828,123 of which the total adjusted contract amount for the Stormwater Facility was $1,321,026. It should be noted that additional direct costs of construction were sustained by all parties concerned in the construction and post-construction evaluation phase. Some of these additional costs are included in this cost analysis because these costs would normally be incurred in the grant-eligible portion of a construction project of this scope, and should, therefore, be included in the total cost analysis of this evaluation.

Table 29 is a breakdown of the cost of construction of the Stormwater Treatment Facility. It should be noted that these construction costs were incurred in 1972, 1973 and 1974.

Table 29. DETAILED COSTS OF CONSTRUCTION OF FORT WAYNE STORMWATER TREATMENT FACILITY

Description	Unit Cost Basis	Construction Cost
SITE DEVELOPMENT AND PREPARATION		
Excavation - 39,614 CY	$1.57/CY	$ 62,029.99
Wetwell Structure Construction	L.S.	394,650.00
Backfill Wetwell Structure & Facility Site Work	L.S.	74,643.31

Table 29. DETAILED COSTS OF CONSTRUCTION OF FORT WAYNE STORMWATER TREATMENT FACILITY (Continued)

Description	Unit Cost Basis	Construction Cost
Subtotal Site Development & Preparation		$531,323.30
STORMWATER PUMPING AND SCREENING BUILDING ERECTION		
Mechanical Trash Rack Installation	L.S.	$ 92,948.00
Pumping & Screening Building Installation	L.S.	60,900.00
Stormwater Pumping & Screening Building Mechanical Services	L.S.	34,922.50
Subtotal Stormwater Pumping & Screening Building Erection		$188,770.50
STORMWATER TREATMENT FACILITY PUMPING EQUIPMENT		
Stormwater Pumps	L.S.	$ 54,056.00
Stormwater Pumps Installation & Start-Up	L.S.	5,200.00
Stormwater Pumps Discharge Piping	L.S.	9,741.00
Wetwell Sump Pump	L.S.	10,695.00
Subtotal Stormwater Treatment Facility Pumping Equipment		$ 79,692.00
STORMWATER FACILITY FLOW DISTRIBUTION SYSTEM		
Head Tank Fabrication	L.S.	$ 10,900.00
Influent Piping to Screening Units	L.S.	69,020.62
Subtotal Stormwater Facility Flow Distribution System		$ 79,920.62
STORMWATER FACILITY SCREENING EQUIPMENT		
Bauer Hydrasieve®		
Equipment Bid Purchase Price	L.S.	$ 60,500.00
Materials & Labor to Mount Bauer Screens	L.S.	14,486.83
Subtotal Bauer Hydrasieve® Equipment, Materials & Labor as Installed		$ 74,986.83
Rex Rotary Drum Screen		
Equipment Bid Purchase Price	L.S.	$ 39,400.00
Materials & Labor to Mount Rex Screen	L.S.	8,600.00
Materials & Labor to Install Rex Screen	L.S.	3,900.00

Table 29. DETAILED COSTS OF CONSTRUCTION OF FORT WAYNE STORMWATER TREATMENT FACILITY (Continued)

Description	Unit Cost Basis	Construction Cost
Subtotal Rex Rotary Drum Screen Equipment, Materials & Labor as Installed		$ 51,900.00
SWECO Centrifugal Wastewater Concentrator®		
Equipment Bid Purchase Price	L.S.	$138,744.00
Materials & Labor to Mount SWECO	L.S.	5,700.00
Materials and Labor to Install Mechanical Services to SWECO's	L.S.	8,200.00
Materials & Labor to Install Electrical and Controls to SWECO's	L.S.	15,950.00
Subtotal SWECO Centrifugal Wastewater Concentrator® Equipment, Materials & Labor as Installed		$168,594.00
STORMWATER FACILITY ELECTRICAL		
Sitework, Service and Grounding	L.S.	$ 8,130.00
High Voltage Switchgear & Transformer	L.S.	19,818.00
Motor Control Centers & Equipment	L.S.	89,788.00
Stormwater Facility Fixtures, Devices, Wire, Conduit & Boxes	L.S.	28,976.00
Subtotal Stormwater Facility Electrical		$146,712.00
STORMWATER FACILITY CHLORINATION		
Water Pollution Control Plant Chlorination Building Modifications		
Chlorination Equipment, Labor & Materials	L.S.	$ 14,400.00
Chlorination Building Piping & Fittings	L.S.	6,000.00
Subtotal Water Pollution Control Plant Chlorination Building Modifications		$ 20,400.00
Stormwater Facility Chlorination Piping & Valves		
Chlorine Solution Line Piping & Valves	L.S.	$ 21,516.39
Subtotal Stormwater Facility Chlorination Piping & Valves		$ 21,516.39

Table 29. DETAILED COSTS OF CONSTRUCTION OF FORT WAYNE STORMWATER TREATMENT FACILITY (Continued)

Description	Unit Cost Basis	Construction Cost
STORMWATER FACILITY MISCELLANEOUS AND UNCLASSIFIED		
Stormwater Facility Safety and Other	L.S.	$ 10,744.87
Subtotal Stormwater Facility Miscellaneous & Unclassified		$ 10,744.87
TOTAL CONSTRUCTION COST OF STORMWATER FACILITY		$1,374,560.51

ANNUAL COST OF OPERATION AND MAINTENANCE

Costs data obtained for the Fort Wayne Stormwater Facility over the operational evaluation period from January, 1975 to February, 1976 were apportioned to each event for which sufficient data were available to warrant removal efficiency comparisons. These data were compared to efficiency parameters studied to attempt to isolate some correlation between operating costs for chemicals consumed, etc., and treatment efficiency. No such meaningful correlations were obtained for the data taken in this program. See the Appendix material to this report for the breakdown of costs used in this portion of the analysis.

Next, the overall yearly costs for treating the total quantity of stormwater were examined. In order to enable the researchers to meaningfully evaluate the treatment facility power consumption, several categories of power usage were identified with estimates made of the proportional usage for each factor. Since the facility was electrically heated, a large heating and ventilating load was noted for the building. There were approximately 6,200 degree-days at 60 degrees Fahrenheit (15.5 degrees Celsius) during the study period. The average temperature in the screening facility was kept at 50 degrees Fahrenheit (10 degrees Celsius) throughout the cold weather after several of the SWECO screens' flow control valves froze up in February, 1975. This power usage was distributed to each screening unit by its overall flow proportion for the evaluation period. Total estimated power usage for heating was 428,000 kilowatt-hours, or the overall power consumption for ventilation was similarly estimated at 2,565 kilowatt-hours. The lighting load was calculated at 30,635 kilowatt-hours, and other miscellaneous power usage items added another 86,935 kilowatt-hours. Raw stormwater pumping accounted for 78,630 kilowatt-hours total. Summing all these power usage requirements resulted in an estimated power cost of $13,703.65, very close to the actual billings of $13,714.02.

Water consumption for the facility by the City Utilities billings showed a net cost for potable water of $205.00 for the year. Consumption was estimated on the basis of apportioning the observed water usage for the Rex and SWECO screens' backwash make-up consumption. This usage totaled an estimated 4.67 million gallons (17.6 million liters) for these screens. An estimated 92 percent of the facility's total usage went for screening equipment backwash water make-up, based on a delivery rate of 150 gpm (570 lpm).

Chlorine usage was recorded for several treatment events. The cost of liquid chlorine in this region has fluctuated considerably in the past few months. As a result of this uncertainty of price, a cost of $0.16 per pound of chlorine used was used to estimate this cost for comparative evaluation purposes.

Operating labor costs for the evaluation period are not considered typical of the anticipated costs of a similar facility because of the tremendous number of separate grab samples obtained for this evaluation program. As a result, only maintenance costs are considered in the evaluation of costs shown in Table 30, following.

Consumed resources utilized in the program such as chemical cleaners, electrical power, potable water, replacement screens and such are considered in the cost analysis undertaken as a part of this program. The total annual treatment costs are used in the determination of the amount of treatment on a cost per million gallons basis. Table 30 is a summary presentation of the overall cost analysis for operation and maintenance costs, excluding the cost of labor to operate the screens.

On the basis of consideration of the operating expenses incurred over the lifetime of the screening units (estimated at 10 years for all units although the Bauer units do not have the same electric motor-driven components as the Rex and SWECO units), the cost per million gallons treated shows the Bauer is clearly half the operating cost of the other two units. Note that the major expense for this facility is heating and ventilating which was necessary for keeping the SWECO pneumatic solenoid valves from freezing.

Table 30. COMPARATIVE ANNUAL OPERATIONS AND MAINTENANCE COSTS FOR THREE SCREENING METHODS

	Bauer Hydrasieve®	Rex Rotary Drum Screen	SWECO CWC® Centrifugal Wastewater Concentrator
Annual Prorated Cost of Pumping Raw Stormwater	$ 417.76	$ 348.15	$ 692.90
Annual Prorated Cost of HVAC Services	2,285.90	1,905.00	3,791.35
Annual Prorated Cost of Lighting	162.76	135.65	269.95
Annual Miscellaneous Prorated Utility Costs	0.00	57.90	1,079.35
Annual Prorated Potable Water Usage	**	43.60	42.70
Subtotal Annual Utilities Costs	$2,866.42*	$2,490.30*	$5,876.25*
Annual Estimated Preventive and Corrective Maintenance Costs	0.00	$ 735.00	$1,428.00
Annual Estimated Replacement Screens Costs	0.00	1,500.00	3,920.00
Total Annual Estimated Costs for Operations and Maintenance	$2,866.42	$4,725.30	$11,224.25
Total Flow Treated in MG	113.5 MG	94.512 MG	188.1 MG
Annual Operating Cost per Million Gallons	$ 25.25/MG	$ 50.00/MG	$ 59.68/MG

Note: 1 MG x 3.785 = Million Liters

*Utility costs are prorated on the basis of flow treated for each screening unit.

**Clean-up only.

REFERENCES

1. "Lake Erie Report", U.S. Federal Water Quality Administration, Washington, D.C., U.S. Department of Interior, August, 1968.

2. Middendorf, W. H. Engineering Design; Boston, Allyn and Bacon, Inc., 1969, p. 286.

3. Master Plan for Sewers, Part One - Relief Sewers, Henry B. Steeg & Associates, Inc., Indianapolis; City of Fort Wayne, Indiana, May, 1969, 135 p.

4. Grant Offer - Large-Scale Demonstration of Treatment of Storm-Caused Overflow by the Screening Method, Statement of Work; Federal Grant No. 11020 GYU Modification to U.S. Environmental Protection Agency, Chicago, June 14, 1972.

5. Screening/Flotation Treatment of Combined Sewer Overflows; Envirex, Inc., Milwaukee. U.S. EPA Project 11020 FED. January, 1972, p. 172.